A PRACTICAL TREATISE

ON

FOUNDATIONS

EXPLAINING FULLY THE PRINCIPLES INVOLVED

SUPPLEMENTED BY ARTICLES ON THE USE OF CONCRETE IN FOUNDATIONS

BY

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SECOND EDITION, ENLARGED

FIRST THOUSAND

NEW YORK

JOHN WILEY & SONS

LONDON: CHAPMAN & HALL, LIMITED

1906

ROBERT DRUMMOND, PRINTER, NEW YORK

PREFACE TO SECOND EDITION.

IN the second edition of this book the first is enlarged by the addition of pages 398–533. The extensive use of concrete in foundations receives special notice, and would alone justify the addition.

Numerous descriptions of important modern structures are given in sufficient detail to make them understood. The index to the addition has been incorporated into the old index and the whole so revised that the numbers now refer to the pages instead of to articles and paragraphs as before.

Thanks are due the *Engineering News* for the free use of its cuts and columns.

It is to be regretted that the work did not receive the final corrections of the author. His premature death shortly after the manuscript was turned over to the publishers left the work of proof-reading and of all revisions entirely in other hands, and doubtless such inaccuracies as have remained undiscovered would have been detected had the proof been revised by the author. However, the proof has been read with care, all references have been closely examined, and it is hoped that the book is as free from errors as could be expected under the circumstances. It was the author's opinion that this edition would make the work completely up to date.

J. E. WILLIAMS.

VIRGINIA POLYTECHNIC INSTITUTE,
January, 1906.

PREFACE.

IN a work on Foundations, theories and formulæ are of little value; therefore but little space is given to the discussion or criticism of either. The more common formulæ are given without any attempt to explain the laws or premises upon which they are based; a few examples are worked out in order to show the actual or relative values of the terms entering into them, and to compare the results with those used in practice. I do not do this either to ignore or underrate the value or importance of theoretical investigations; if the formulæ deduced in themselves do not have practical value, they incidentally lead to comparisons with actual results, and induce the publication of a large mass of more or less accurate records of observed facts. Theory and practice should go hand in hand; but it is to be regretted that in many institutions claiming to be schools of engineering so great a preponderance in time and energy is given to the theoretical side of the question, even almost to the exclusion of practical instruction, whereby many erroneous ideas and principles are instilled into the minds of young engineers, to eradicate which years of labor, blundering, and mortification are required, causing loss and delays to their employers, loss and injustice to contractors by onerous and useless requirements and exactions, which could have been saved by a knowledge of a few facts and methods found in common and every-day practice; theorists claiming that the "costs of labor, materials and construction, and also rules of practice" are of no value to the student of engineering, as these will be acquired after leaving college, and that "principles alone are necessary to be taught."

Having been a professor for over six years, I have fully realized the need of suitable books, by which I could temper the almost painfully scientific, abstruse, and purely theoretical books that I was compelled to put into the hands of the student in engineering, which could only be partially supplemented from a few years' prior experience in active practice, during which the full force of what I have stated above was fully realized.

With the above experience and the experience derived from eighteen years of active practice, a very large portion of which was devoted to bridge construction in many parts of the United States, building on a great variety of soils, necessarily requiring a great variety in the designs and methods of construction, I have undertaken to write the following pages. The descriptive portions of this volume have been to a large extent based upon my own experience, the facts of which are taken from records made at the time and still in my possession; they can therefore be relied upon as accurate. The drawings, with few exceptions, are taken from my own designs, and are accurate representations of the actual structures used; in these my only aim was simplicity in design, convenience in construction, combined with cheapness, strength, and suitableness for the purpose in view. Unusual sizes and shapes of the parts were studiously avoided, as only adding to the cost of material and construction without any compensating practical advantages. I have only given prominence to these, as I believed they can be fairly well taken as typical designs, and with a few modifications in the details can be readily converted into the designs of other engineers for the same purposes. Full descriptions, however, have been given of all of the latest and largest structures, which can be readily understood when taken in connection with the drawings given. I have collected from all available sources facts in connection with this all-important subject that have been published up to the present date, such as the actual loads and pressures on every variety of material, accurate descriptions of all designs and methods of construction, all useful knowledge of the qualities, properties, and strength of the materials used. Believing that the want of familiarity with the costs of materials and construction, the usual dimensions and forms of parts, and the quantities of materials required in the more common structures, as expressed in bills of material and records of actual and comparative costs of structures, is a most fruitful soures of waste of money in making contracts, as designing contractors, by magnifying the costs of materials and construction, and the difficulties and risks to be incurred, impose upon the credulity, ignorance and fears of engineers, thereby securing enormous profits on their works, for these reasons I have devoted more than the usual space to these matters. I have expressed opinions, made suggestions and (I hope) kindly criticisms, knowing full well that if they are erroneous or not justified by the facts presented they will be corrected, for which kindness I desire to express my thanks in advance. No one need be misled by opinions, as the facts are present in full. I have endeavored in writing this volume to confine myself as closely as possible to matters pertaining to the subject of foundations, by which I mean those parts of structures resting on and directly supported by the materials of the earth, and these materials themselves in regard to their capacity to support the loads or pressures resting upon them. There has always been some confusion as to the meaning of the term Foundation: it is difficult, if not impossible, to separate that which supports a pressure from that which produces it. We must know the magnitude, the direction, and the point of application of a force; and all three must be known. If the force is distributed, we must know the nature of the distribution, whether uniform, uniformly varying, or irregularly varying, so as to provide proper supports and resistances, with the requisite strength and at the required points. Except in so far as these considerations enter, I think that I have confined myself within the limits of the subject. To avoid confusion or too much repetition, I will always call the natural materials, of whatever nature, upon which the structure is founded or built, the Foundation-beds; all else will be called Foundations or Substructures, these being the parts of the structure under the surface of the ground or water. Those portions above are only described or illustrated where it could not be avoided either for a clearer understanding or for sake of valuable comparisons. Where tables and other data have been taken from books, I have endeavored to give the authors the credit in the description. I am, however, largely indebted to the editors of the *Engineering News*, who kindly granted the free use of the columns of their valuable and wide-awake magazine. I am also under obligations to Mr. C. A. Brady. C.E.; Mr. I. E. A. Rose, architect; and Professor R. A. Marr, of the Virginia Military Institute, for valuable aid in preparing the drawings. I have been greatly assisted in other ways by Col. E. W. Nichols, Prof. of Mathematics, Virginia Military Institute.

The volume is divided into three parts; it is further subdivided into articles and paragraphs. The articles are numbered continuously throughout the volume, the paragraphs are only numbered continuously through each part. Par. 1 is at the beginning of each part.

W. M. PATTON, C.E.

LEXINGTON, VA., May, 1893.

TABLE OF CONTENTS.

PART FIRST.

ARTICLE I.

ARTICLE II.

ARTICLE III.

ARTICLE IV.

ARTICLE XXVII.

ARTICLE XXVIII.

PART SECOND.

ARTICLE XXIX.

ARTICLE XXX.

ARTICLE XXXI.

PART THIRD.

ARTICLE XLVII.

ARTICLE XLVIII.

ARTICLE XLIX.

ARTICLE L.

ARTICLE LI.

ARTICLE LII.

ARTICLE LIII.

ARTICLE LIV.

ARTICLE LV.

ARTICLE LVI.

ARTICLE LVII.

SUPPLEMENT.

PAGE

LIST OF FIGURES.

A PRACTICAL TREATISE ON FOUNDATIONS.

PART FIRST.

ARTICLE I.

FOUNDATION-BED.

1. NOTWITHSTANDING the almost infinite variety of materials upon which we have to build and do build, there are certain general principles that should be followed, and which are applicable in all cases.

2. First. The surface of the foundation-bed, excepting where piles are used, should be perpendicular to the direction of the resultant pressure, i.e., horizontal in case of ordinary bridge piers, walls of houses, and in general, in all cases where the resultant pressure is vertical; and in fact in cases where the resultant pressure is inclined to the vertical—as in case of retaining-walls, a horizontal foundation-bed will usually prove to be safe. This does not mean that on solid rock the foundation-bed must be cut over its entire surface to one horizontal surface, or even cut into a series of horizontal surfaces resembling steps,—this costs a great deal of time and money,—but that the surface of the foundation shall be so roughened as to

prevent the possibility of the substructure slipping on the foundation-bed. Illustrated by the following diagrams:

FIG. 1.—LONGITUDINAL SECTION OF FOUNDATION-BED ON ROCK.

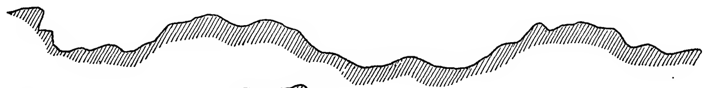

FIG. 2.—TRANSVERSE SECTION OF FOUNDATION-BED ON ROCK.

This is especially applicable to a foundation-bed of rock. In all other materials a uniform horizontal surface or a series of steps will be found both convenient and economical. And in fact in rock a series of blast over the surface, making a number of irregular depressions, will satisfy all conditions of safety.

3. Second. An excavation must be made for a certain depth, depending mainly upon the depth to which alternate freezing and thawing takes place; this depth—say from (2) two to (6) six feet—depending upon the climate and latitude, but may be limited in rock to removing loose and disintegrated portions.

4. Third. As far as possible, surface water should be excluded from the foundation-bed, and all possibility of running water should be absolutely excluded. This is accomplished by surface drains, and where necessary by subsoil drains.

The principles above stated are applicable to rock, clay, sand, gravel, and various combinations of the three latter.

5. Fourth. Uniformity of material in the foundation-bed is absolutely necessary. It is almost certain that any kind of material, except rock, will settle more or less under pressure, and will settle irregularly, consequently the structure will inevitably crack somewhere. Build wholly on one or other of the materials mentioned.

6. Fifth. The weight of the structure should be as uniform as possible, and the structure should be built on all sides as

nearly of the same height as possible. If heavy towers, such as the spires of churches, and they are bonded at all to the body of the building, special provisions (hereafter described) should be made so as to make the unit pressure (pressure per square foot of foundation-bed) the same as under any other part of the structure.

7. The above principles being followed, safety against slipping is fully provided, and partly against settling. But another important element is the unit weight or pressure per square foot of structure upon the foundation-bed. Our knowledge as regards the capacity of bearing weight is meagre, and such as we have is conflicting and uncertain. The test of a cubical block of stone 2 in. × 2 in. × 2 in. of 4 sq. in. of surface, with cushions of pine, lead, or other substance, under pressure, can scarcely be considered as determining the crushing resistance of immense volumes of the same material in quarries or when built into massive structures, as valuable as it may be in other respects. But even the strength thus attained is sufficient to carry any load liable to occur in practice.

8. To illustrate: the most reliable authorities give the resistance to crushing of weak sandstone 3000 lbs. per square inch; a granite pier 180 ft. high, carrying one half of two spans of 525 ft. length and a rolling load of 3000 lbs. per lineal foot, gives a resultant weight of only 150 lbs. per square inch, giving a factor-of-safety of 20. Therefore we may conclude that almost any structure that we are likely to build can be safely constructed on the three types of rock commonly met with—granite, limestone, sandstone.

9. Some authorities class clay, sand, sand and gravel together, and state that 3000 lbs. per square foot of foundation-bed is the greatest intensity of pressure admissible. The writer, however, gives to clay the precedence, for the following reasons: Clay is more compact; along with its tendency to retain water it has an equal power of excluding water; if settlement takes place it is apt to be uniform under same pressure, and consequently is less liable to cause damage to structure above;

it does not scour, and the weight on such material has a tendency to aid in keeping water out of that space over which weight is distributed. Water can be more easily kept from the foundation-bed either by surface or subsoil drains. The above authorities do not state the exact quality of material alluded to, as clay may vary from a soft, pliable clay, through loam, a mechanical mixture of clay and sand or what might be called "brick clay," and marl, a mechanical mixture of carbonate of lime and clay, together with certain silicates and protoxide of iron, then culminating in what may be called an indurated clay.

10. If the low unit pressure, 3000 lbs., is the limit of safety, but relatively small structures should be built upon it without taking unusual precaution to distribute the pressure over a large area or by compacting the material by the use of piles. Taking the weight of a brick wall at 120 lbs. per cubic foot, the material above mentioned would only bear a column of masonry 25 ft. high and 1 sq. ft. base; but this weight or pressure can be easily distributed over two, three, or more square feet of foundation-bed. So for any ordinary structure the above limit of resistance need not be exceeded, and in view of the fact that so many structures do settle and often cause dangerous cracks, it is unwise to take any risk. The writer built a bridge across the Ohio River at Point Pleasant, W. Va., on what he has classed an indurated clay—evidently a clay containing carbonate of lime. It could be worked into a paste with water. Frequently the pit would be flooded. After pumping out the water a thin layer of slush or paste would be found. When this was scraped off, to the depth of an inch, rarely more, the surface was as dry and as hard as before. The largest pier was about 100 ft. high, carrying one span of 400 ft. and another of 200 ft., built of sandstone, producing approximately a pressure of 5000 lbs. per square foot of foundation-bed, assuming sandstone at 150 lbs. per cubic foot and spread doubling area of base. These, then, can be taken as the safe limits for a clay foundation. Some clays have seams in them, generally sloping at a greater or less angle to the vertical: these, if extensive, are dangerous, as the water will percolate along them, causing a dangerous tendency to slide. In these cases the water must be excluded or the depth cut into material greatly increased.

11. Building on sand was pronounced dangerous in the Bible, and has been so considered ever since; but circumstances often compel us to build on this tempting material, and as it may be said take the chances. Sand, when confined, is considered practically incompressible within the limit of actual crushing the grains of sand into impalpable powder. Sand will hold your structures if you can hold the sand. But here is the difficulty: it is porous, and unless confined in walls of rock or clay there is always danger of the water passing through, scouring out the material, and undermining the foundation, this process being greatly aided by the weight of the structure, and sometimes forming with water quicksand, which is almost as unstable as water itself. Therefore in building on sand under no circumstances exceed the limit of weight of 5000 lbs. per square foot of foundation-bed, and in addition be sure of excluding the water, or in exposed situations drive piles. More on this point hereafter. Beds of gravel and bowlders especially can certainly be relied upon, to at least the superior limit for clay of 5000 lbs. per square foot of bearing surface. Two of the high piers of the Susquehanna River bridge, B. & O. R. R. at Havre de Grace were built on bowlders large and small, but at a great depth below the bed of the river, in which the frictions on the sides supports much of the weight.

12. The remaining material of silt or slush, such as we find in all the swamps, especially in the Southern States, can scarcely be made safe without the use of piles, for very heavy structures; but by the liberal use of broken stone, or even in some cases of sand or gravel, a reasonably stable foundation-bed may be artificially constructed, which will be fully explained further on. The above sets forth fully the actual and relative merits of foundation-beds generally met with in actual practice. All of these matters will be incidentally alluded to when we come to discuss foundations, which is the next division of the subject to be treated.

13. A combination of these materials is frequently met with, the bearing-power of which may practically be taken the same as above; but frequently these combinations take the form of what is called hardpan, or cemented sand and gravel, that would certainly justify a higher classification, and would not be inferior to the ordinary kinds of rock. There is no mistaking this material when met with, and it can be relied upon to bear the weight safely of any ordinary structure. In many of the Southern States there is an earthy substance which may be called a marl, easily cut into blocks, difficult to excavate, requiring blasting sometimes, and capable of bearing heavy loads, but disintegrating rapidly when exposed to the air, and consequently unfit for building purposes. The writer founded several piers on this material, carrying long spans 275 ft. in length; the piers were of brick and the pneumatic caisson used, passing through sand and silt before reaching it. This material almost disintegrated by slacking when exposed; it effervesced freely with acids.

14. The practical deductions from the above, then, may be stated as follows:

1st. That it is in general perfectly safe to build, on any material that can be called rock, any structure likely to be required.

2d. Bowlders and gravel can also be considered perfectly reliable for any ordinary structure under any ordinary conditions. Scour alone should be guarded against, which, however, is not probable.

3d. Sand is safe to bear a load of any amount, provided it is confined; but great precaution must be taken to confine it, and also keep water, especially running water, from it.

4th. Clay, when compact and dry, will likewise carry very large loads. Water should be kept from it both under and around the structure, as it may give way if it gets in the condition of paste by bulging up around the structure.

5th. In the last three cases the base of the structure should be so spread out as to keep the pressure per square foot of base within the safe limit, and the depth below the surface must be below the action of frost, which varies from 2 ft. to 6 ft.; and in soft kinds of material the deeper the better.

15. A thick, hard, or compact strata overlying a much softer one, even silt or quicksand, will often carry a considerable load, the hard strata as it were floating on the softer. It is sometimes better not to break through it, as it has the effect of spreading the base and distributing the pressure over a large area. Good judgment is here required, and some risk must be run. This principle is followed when planks or logs are spread out on the soft material, and the structure built on the logs, the logs forming a broad bearing surface. Mr. Rankine states that Chat Moss was crossed by the use of dry peat and hurdles or fascines in layers forming a raft, which carried a railway on it. It would seem safer and more satisfactory in such cases to drive piles.

16. The following figures give the actual bearing-power of some of the above materials.

Mr. Rankine says, page 361, "Civil Engineering:"

Granite	12,861	lbs. per square inch.
Sandstone	9,842	" " " "
Soft sandstone	3,000 to 3,500	" " " "
Strong limestone	8,528	" " " "
Weak limestone	3,050	" " " "
Clay, sand, and gravel	17 to 23	" " " "
Brick	1,100	" " " "

And gives the actual pressure on some existing foundations as only about 140 lbs. to the square inch, giving an actual factor-of-safety of about 22, whereas factor-of-safety from 8 to 10 is considered ample. These are probable average values.

17. Mr. Baker, in his treatise on Masonry Construction, page 10, gives the following as the crushing strength of stone:

Granite from 12,000 to 21,000 lbs. per sq. in.	= 860 to 1,510 tons per sq. ft.
Marble from 8,000 to 20,000 lbs. per sq. in.	= 580 to 1,440 tons per sq. ft.
Limestone from 7,000 to 20,000 lbs. per sq. in.	= 500 to 1,440 tons per sq. ft.

Sandstone from 5,000 to 15,000 lbs. per sq. in.	= 360 to 1,080 tons per sq. ft.
Brick from 674 to 13,085 lbs. per sq. in.	= 48 to 936 tons per sq. ft.
Clay from 28 to 84 lbs. per sq. in.	= 2 to 6 tons per sq. ft.
Gravel from 112 to 1,401 lbs. per sq. in.	= 8 to 10 tons per sq. ft.

The above doubtless gives the results of the latest experiments. There are special cases when the loads actually borne are greater than the above; but we can safely conclude that good ordinary clay will carry safely two tons per square foot; sand, from 3 to 4 tons to the square foot, provided it can be kept entirely free from water.

18. In cases of doubt and the absence of precedent, when unusually heavy loads are to be carried, and especially when the weight of the structure is not uniformly distributed, as in case of high towers and spires, tests should be made by actual weights placed on a unit of area, which can be done at the cost of but little time and money; and as the means are always in reach to make the foundation safe, it is certainly inexcusable, to say the least of it, to blunder along and take the chances of the structure falling, involving great loss of property, if not of life, and only to avoid expending a few dollars.

19. When structures fail, it may in general be said that it is impossible to determine the cause, though in general it is easy to get numberless opinions of so-called experts, and with these the public and juries are satisfied; but in a large majority of cases it can be traced to that part of the structure under ground or under water, and ultimately due to the failure of the foundation-bed: for even if the part of the structure under ground is defective in some of its parts, it throws an excessive weight on some part of the foundation-bed. The failure, from high winds, from thrusts of roof or floors, or from floods, drift, and ice, is generally indicated by the manner of the falling; and though this may evidently be the direct cause of failure, yet, indirectly the foundation-bed is at fault, as these cause undue pressure on some parts or scour out the foundation-beds and undermine the structure, as other conditions and requirements always require such weights and sizes of structures as will resist the outside forces. The dimensions of bridge piers are regulated generally by the dimensions at the top required as a rest for the bridge structure, and are greater than that necessary to withstand the effects of these external forces. Be sure of your foundation and foundation-beds, and except in extreme cases the upper part of the structure will take care of itself.

ARTICLE II.

FOUNDATIONS.

20. THIS division of the subject includes that part of the substructure reaching from the foundation-bed to the surface of the ground or the surface of the water, and necessarily includes the various means of reaching the foundation-bed, such as ordinary excavations on land, driving piles on land or in water, screw-pile foundations, Cushing cylinders, coffer-dams, pneumatic cylinders, pneumatic caissons, open caissons, pierre-perdue foundations on land or in water, sand foundations in swamps, concrete foundations, rubble-stone foundations, etc.

21. Each of these divisions will be treated more or less elaborately, but purely from a practical standpoint and as concisely as the importance of the subject may demand, consistently with that amount of detail as may be necessary to a clear understanding of the matter. These will also be accompanied by drawings giving sufficient details to be of actual and practical use. Many books mystify with useless formulæ, and give just enough practical information and details as to leave you in doubt whether you know anything at all, as it is generally admitted that in many cases the formulæ have no practical value. This the writer hopes to avoid, and at the same time not to extend the limits of this subject too far.

CONCRETE.

22. As concrete is used so extensively, and in combination with almost all kinds of foundations, we will commence with this material.

Concrete is composed of broken stone or gravel or both, sand, cement, and water, mixed under certain circumstances in absolutely definite proportions, so as to obtain a conglomeration which experiments, conducted principally by Government engineers, have shown ultimately to produce the best possible results; and doubtless in all works this practice would be followed, if in works paid for by private individuals or companies, we had the money and time at our disposal. But in works of this class we must aim to attain as near perfection as practicable, but be satisfied with what is good enough for the purpose in view, with the least possible cost in time and money, consistent with securing a permanent, strong, safe, and durable structure. We will first, however, explain the process of making concrete in accordance with the requirements of the Government engineers.

23. Gen. Q. A. Gillmore's treatise on Limes, Hydraulic Cements, and Mortars is assumed to be high authority,—a book which contains valuable and interesting information. On page 226, paragraph 450, we find: "The concrete was prepared by first spreading out the gravel on a platform of rough boards, in a layer from eight to twelve inches thick, the smaller pebbles at the bottom and the larger on top, and afterwards spreading the mortar over it as uniformly as possible. The materials were then mixed by four men, two with shovels and two with hoes; the former facing each other and always working from the outside to the centre, then stepping back and recommencing in the same way, and thus continuing the operation until the whole mass was turned. The men with hoes worked each in conjunction with a shoveller, and were required to *rub well into the mortar* each shovelful as it was turned and spread, or rather scattered on the platform by a jerking motion. The heap was turned over a second time in the same way, but in the opposite direction; and the ingredients were thus thoroughly incorporated, the surface of every pebble being well covered with mortar. Two turnings usually sufficed to make the mixture complete, and the resulting mass of concrete was ready for transportation to the foundation."

There is but little comment to make; the method for hand-mixing can be safely recommended. The writer has mixed large quantities in practically the same manner, with these modifications: Firstly, the broken stone or gravel was not screened so as to separate the larger from the smaller sizes, and place the smaller pebbles at the bottom and the larger on top. The broken stone or gravel, within special limits as to the large size, the limit being such as would pass through a 2½-inch ring, determined by inspection, and used the material as delivered; and secondly, that no hoes were used, all the men using the shovel as described, and each shoveller as he turned over his shovelful made three or four cuts into the mass with his shovel in a nearly vertical position, the object being to ram the mortar between and over the broken stone, and also prevent the mass from being heaped up, which would cause the stone to roll down to the base of the mass, and leaving a surplus of mortar on top. This operation was continued until every stone was covered. Mixing by hand is rarely economical or rapid enough where large quantities of concrete are to be made in a limited time. The method of mixing mortar, together with the ingredients and proportions of the same, whether mixed by hand or machinery, are elaborately explained in Gen. Gillmore's treatise, pages 192 to 206 inclusive, to which for valuable information the reader is referred.

24. The consistency of the mortar—whether very soft, in a pasty condition, or almost dry—is not explained. This is an important consideration, and one upon which there is a wide difference of opinion. In the Appendix to Gen. Gillmore's treatise he speaks of the mortar as being "about the consistency of plasterer's mortar." In an extended experience the writer of this work has found this consistency to give the best results in many ways, can be more readily incorporated, as well as more uniformly mixed; can be handled more readily; can be compacted by ramming without producing a spongy, springy mass; takes its initial set more readily; and certainly for ordinary purposes is more satisfactory than when the mortar is more liquid, as well as when it is too dry and stiff. In the Ap-

pendix Gen. Gillmore gives some valuable information on the cost, qualities, and proportions of ingredients of concrete on Staten Island, which is well worth studying, as well as methods of mixing mortar and concrete by hand and machinery. Only one or two tables of proportions will be given.

"Concrete No. 1:

1 bbl.	German Portland cement,	5.4 bbls. concrete mortar.
$5\frac{1}{2}$ "	damp sand loosely measured,	
6 "	gravel and pebbles from sea-shore,	12 bbls.; 26 per cent of voids.
9 "	broken stone,	

Producing 50 feet of rammed concrete. This concrete is of first-rate quality, being compact, free from voids, and strong. It is richer in mortar than would be necessary for most purposes." Proportions 1 mortar to $2\frac{1}{2}$ stone and gravel—evidently a large excess of mortar over quantity necessary to fill voids. In the writer's experience 2 barrels of sand to 1 barrel of cement for ordinary and 3 barrels of sand to 1 barrel of Portland cement seem to be the rule for use in constructing foundations for bridges of great magnitude. For less important work from 4 to 5 of sand to one of cement.

"Concrete No. 5:

1 bbl.	Rosendale cement,	3.27 bbls. concrete mortar.
3 "	damp loose sand,	
5 "	broken stone,	

Will yield 21.75 cu. ft., rammed in position." This mortar possesses a crushing strength of 130 lbs. per square inch when two months old. "Another proportion given:

4 barrows of mortar (8 cu. ft.);
6 heaped-up barrows of broken stone (14 cu. ft.);
6 heaped-up barrows of gravel (14 cu. ft.)."

This would seem a good proportion of ingredients. No mention is made of the resulting quality of concrete.

25. It will be observed from the proportions above given that Government engineers seem to prefer an admixture of gravel with the broken stone,—presumably to save mortar. It is rarely the case that gravel and stone can be economically secured at the same time, and consequently as a rule only one of these elements can be used; and when it can be done the chances are that one part of the concrete will be largely of stone and the other largely of gravel, as there is no known law by which gravel can be forced to place itself between the pieces of stone. Either alone makes good concrete, as doubtless a mixture of the two will; but many would prefer the angular and rough broken stone to round and smooth gravel, provided the stone is as hard as granite or limestone, or some of the varieties of hard sandstone. Broken bricks and shells are often used in localities where neither stone nor gravel can be found; gravel would evidently be preferable to brick or shells. The mortar takes hold of the broken stone, thereby tying and binding the whole mass together, which does not take place when gravel is used, as can easily be seen by breaking a block thus made: the gravel pulls away from the matrix or mortar, leaving round, smooth holes. For most purposes concrete has only to bear a crushing strain, and is not subjected to a tensile strain unless a foundation is undermined, which ought not to occur often.

26. The writer has used over 30,000 cu. yds. of concrete, supervising to a considerable extent the mixing, in all its details, personally; but owing to the circumstances under which he was placed, it was impossible to give that particular attention to exact proportions as might conduce to the very best results, but certainly good enough for the purposes intended, as it has stood for years bearing enormously heavy steady loads, and the heaviest known rolling loads running at the highest speed: therefore he can say that he has fully complied with all the conditions of good work, strength, durability, safety, with the least cost and time; and even permanency can safely be claimed. He will therefore give the benefit of his experience on such bridges as the Ohio River bridge, the Susquehanna and Schuylkill River bridges on the B. & O.

R. R., and the Tombigbee River bridge in Alabama, each of which will present some difference.

27. Taking them in order. We used concrete resting on indurated clay; there were four piers resting on the concrete. The mortar was 2 sand, 1 cement, composed of a fair average sand, clean and sharp; the cement used was known as the Louisville cement. These were mixed by hand in proportions of 1 cement and 2 sand, water sufficient to form a paste of the consistency of plasterer's mortar; the sand and cement were thoroughly mixed dry by turning over and over with shovels. This mixture was then formed into a circular dam, and a small quantity of water was poured inside; a portion of the dry mixture was pulled by hoes towards the centre and thoroughly mixed with the water, care being taken not to let the water escape, as it would carry the cement off. If this mixture was too dry, more water was added and thoroughly mixed, and this process continued until the entire batch was of the proper consistency. The broken stone was a hard bluish-gray sandstone found near by, and small enough to pass through a ring 2 inches in diameter—as close as could be expected. A thin layer of this stone was spread on a platform, upon this a layer of mortar, on top of which another layer of stone, and then another of mortar; this was then turned over and over with shovels as previously described, until every stone was coated with mortar; and it presented a uniform appearance of mortar and stone mixed. On this work the mixing was generally done in the foundation pit, and the concrete was then thrown with shovels into layers of about 10 to 12 inches thick, and rammed in place. A pine plank, 3 inches thick by 12 inches broad, cut in the form of a rammer, seemed to serve the purpose better than a round heavier rammer, suggested by ramming clay puddle. The ramming was continued until a thin skim of water appeared on the surface, then another layer of concrete was put on top of this. Under some of the piers clean river gravel was used instead of broken stone, mixed and compacted in place as above, with equally satisfactory results. The proportions were usually 1 barrow

of mortar to $2\frac{1}{2}$ barrows of stone, varied somewhat as the size of the stone varied. With a little experience the proportions would be easily adjusted by the eye, the aim being to have all the interstices filled. The broken stone was moistened. We secured a reasonably uniform result. The quantity here was not very great—649 cu. yds.

28. At the Susquehanna and Schuylkill River bridges all the concrete in the cribs above the caisson roof was mixed by machinery. All that portion of the concrete in the working chamber of the caisson was mixed by hand, as above described, only small quantities being used at a time. The concrete for the crib was mixed as follows: A revolving drum with buckets, similar to those on an overshot water-wheel, proportioned so as to carry 2 or 3 of sand to 1 of cement, fed through two distinct hoppers, dropped, as it revolved, the sand and cement into a trough in which was placed a revolving worm-screw about 10 feet long; the sand and cement were carried around and forward, thoroughly mixing them dry; at a certain point, determined by experiment, water was admitted from a spigot; experiment determined how much was necessary to be admitted. Water, sand, and cement were now turned over and carried forward; everything was so adjusted that at the end of the trough a paste of the proper consistency was found (this apparatus was the invention of Charles Sooysmith, one of the contractors). At the end of the trough the mortar dropped into the concrete mixer, which can best be described as about two thirds of an iron cylindrical pug-mill, 6 or 8 feet long, gently sloping downwards from the end of the trough, the arms of the revolving shaft in the mixer being so set as to cause the materials in the mixer to be revolved over and over and at the same time moved forward. The proper proportion of the broken stone to a barrel of cement having been collected near the upper end of the mixer, it was shovelled into the mixer as the mortar dropped in from the trough. Intelligent men soon learned to shovel at a uniform rate, and would commonly throw in with reasonable approximation the proper proportion of stone to mortar delivered. The concrete by the time it reached the lower end of the mixer was thoroughly mixed, and then dropped into wheelbarrows and carried to the place of deposit. There were defects in this method. Absolute uniformity was not obtained, but even then we had a remedy: if the concrete when it dropped into the barrows was too wet or too dry, or had a larger proportion of stone than the mortar could carry, or not thoroughly mixed, it was thrown away, and the proportions readjusted. Some waste resulted; some little time was wasted. The proportions aimed to be used were as 1 of mortar to $2\frac{1}{2}$ of broken stone. The concrete for these structures was generally dropped from a greater or less height, as the timber work was always built well ahead of the concrete; but nevertheless it was distributed in layers with the shovel, and rammed as before described.

29. The stone at the Susquehanna was granite, at the Schuylkill limestone, broken in both cases by the Gates crusher. No attempt was made to screen the stone; the impalpable dust to a large extent was blown away; but the stone as it came from the crusher was delivered at the caisson, and consisted of stones say from 3 inches in diameter through all sizes down to the size of coarse sand: this was taken into consideration in proportioning the sand in the mortar. The broken stone was generally kept moist, always in hot weather. In the crib of one of the piers at the Schuylkill, as a matter of economy, the crib was filled with what may be called rubble-work, one-man stones being simply imbedded in mortar. Great care is necessary in this kind of work to secure a solid, compact structure, and there is danger of great waste of mortar; but why it should not be as good as concrete in large masses is probably hard to explain, as to some extent it does look like folly to break stones up simply to cement them together again: but good practice does seem to lean towards concrete. At both of these bridges large stones (one-man stone) were placed at intervals on the surface of a layer of concrete and then covered over with another layer, of which, however, the writer doubts the wisdom. It may do no harm, but surely it does no good: it would not lessen the cost or the time. All concrete or all rubble is best.

30. As to the Tombigbee River bridge, located in the almost limitless swamps of Alabama, there was nothing especially notable, except its inaccessibility, and the almost total absence of building material of any kind, except we may say good pine timber. It is true a limited amount of gravel could be found, but this mixed with sediment from the frequent overflows. Good sand could be found in places; the gravel had to be washed. We were compelled therefore to use broken brick, which had to be brought from Mobile on barges a distance of over a hundred miles; a small quantity of broken stone left there by incoming vessels, which had been used as ballast; consequently oyster-shells brought, by schooners hundreds of miles distant, from oyster-banks had to be relied upon, and this had to be provided and delivered at high stages of the water. The mixing was by hand as previously described; there was nothing new or novel, except materials used. These materials for concrete are the last resort of engineers, and of the two broken brick is the best. But much can be done with good cement and clean sharp sand.

31. The following general principles should be observed in making concrete:

Use good cement and clean sharp sand for the mortar, in proportions, depending upon the quality of the cement, of 2 to 4 of sand to 1 of cement; sufficient water to produce a somewhat soft and plastic paste.

Use the hardest stone available, granite, limestone, hard varieties of sandstone, gravel, etc. This to be broken as nearly as practicable so as to pass through a ring of 2 inches in diameter. Moisten the stones certainly in hot weather. Use somewhat more mortar than is necessary to fill the voids, which will depend upon the size of the stone, whether broken by hand or machinery, also upon the material; but in general from 2 to 4 volumes of broken stone to 1 of mortar. Mix thoroughly the sand and cement, and mix thoroughly the mortar and stone.

Deposit the concrete in layers of not over 12 inches in thickness, and ram until a thin skim of water appears on the surface.

Mortar scarcely moistened is recommended by some engineers as producing ultimately the best result.

31½. In a letter from Gen. T. L. Casey, U. S. Engineer, the proportions of cement, sand, pebbles, and broken stone for the concrete sub-foundation of the Washington Monument were given as follows:

"1 volume of cement, dry;

"2 volumes of sand, clean, sharp, and medium size;

"3 volumes of pebbles, clean, and varying in size from a buck-shot to pigeon's egg;

"4 volumes of broken stone, clean, and small enough to pass through a 2-inch ring.

"A 'batch' consisted of ¾ barrel of cement, 1½ barrels of sand, 2¼ barrels of pebbles, 3 barrels of broken stone. In dry weather about 10 gallons of water was used to a batch, but in wet, soaking weather no water was added. The ingredients were mixed in a cubical box 4 ft. on each edge, rotating on a diagonal axis passing through the box. The mixer was turned eight times for each batch. The concrete when emptied from the box was about as moist as moist brown sugar. Three of these batches made a cubic yard. It required 1⅛ bbls. of cement per cubic yard of concrete. Cost per cubic yard concrete, $6.56."

This concrete was very dry. The writer tried these proportions at the Susquehanna, except the pebbles, but found the concrete too dry to handle in large quantities and rapidly in a satisfactory manner, failed in getting the stones uniformly distributed when rammed in place, and after waiting for several days after depositing the concrete in the crib, found that no change whatever had taken place; the sand and cement were still dry and separate, no set whatever had taken place, and the condition of the mass was the same as if broken stone was simply mixed with so much sand. After this more water was used, which seemed to be very much more satisfactory, both as to setting and ease of handling and compacting into a homogeneous mass.

32. The proportions of cement to sand, and the proportions

of mortar to broken stone or stone and gravel mixed, seem to vary in the practice of engineers between wide limits, and all apparently produce satisfactory results; economy doubtless in most cases being the most potent factor, but necessarily controlled by the size of the stone used and the manner of breaking it, whether the stone is screened or not, and the importance and magnitude of the structure. In the first caisson sunk at Havre de Grace the stone was screened, using only the stone of considerable size. According to records kept, we used 2283 barrels of cement and made 1979 cu. yds. of concrete: this includes the large one-man stone used, the whole estimated as concrete, or 1 barrel of cement made only about 0.9 cu. yd. of concrete; whereas the average of the other four caissons, the stone not being screened, the average yield per barrel of cement was 1.15 cu. yds. concrete. The entire work consumed 14,288 bbls. of cement, mainly Portland, and yielded 14,966 cu. yds. of concrete, or practically 1 bbl. of cement to 1 cu. yd. of concrete. The unscreened stone resembles closely the mass of broken stone mixed with gravel, and requires proportionately less mortar.

33. In handling large masses of concrete an absolute rule as to proportions would hardly lead to anything more than approximately uniform results, as the same crusher will vary materially in the size of the stone broken from day to day, but with the same stone broken under the same general conditions the variation might not be material. A simple method of determining the volume of voids in a cubic yard, such as filling a box containing one cubic yard of the stone, after allowing the stone to be soaked with water, then pouring in water sufficient to fill the voids: this volume of water gives the volume of mortar required to fill the interstices between the stone, to which a liberal excess should be added, as it is better to have too much than too little mortar. In some cases mortar alone is used to fill a crib. This is expensive, and to save money, mortar is thrown down in layers, and while in this condition large stones are simply thrown down upon it at random and then another layer of mortar, and so on. This necessarily fails

to produce a homogeneous mass, and unless the stones are carefully placed they will rest on each other, forming open spaces.

ARTICLE III.

USE OF CONCRETE.

34. THERE is such a great variety of purposes to which concrete can be applied, that the principal ones alone will be mentioned. It is used to a large extent and almost exclusively for those parts of the substructure under ground and under water, in masses varying from 2 feet to 40 feet and more in thickness.

In a subsequent article the uses of concrete will be more fully illustrated. The ease with which it is applied, the ease with which it can be made to conform to the irregularities of the foundation-bed, filling in under and around the irregularities, thus avoiding unnecessary blasting, hammering, etc., furnishes the simplest and most satisfactory means of spreading the base of the foundation, so as to reduce the unit pressure on the foundation-bed, and furnishing a uniform surface upon which to build walls of houses, piers, abutments, and other structures; also forming water-tight floors and walls for cellars, lining reservoirs, cisterns; the entire walls of houses can be built of it, and even entire piers, or filling in piers faced with masonry, iron, or timber. In all of these cases it is in general more economical than rubble and brickwork, and certainly far superior to brickwork under ground or under water. Under walls of houses it is commonly not used in layers of over 1 to 2 feet in thickness, mainly to secure a base wider than the body of the wall in order to distribute the pressure over a greater area. Mr. Rankine, in his Civil Engineering, states that the limit of this widening depends upon the depth of the concrete, viz.: Take a wall of a house 2 feet broad at the base and 20 feet long, this would give 40 square feet of bearing surface if built directly on the foundation-bed, but by putting 2 feet of concrete and then building the wall you can extend this concrete 2 feet on each side of the wall, forming a base 6 feet broad and giving a bearing surface of 120 square feet; if 3 feet thick a bearing surface of 160 sq. ft.; and so on. Upon this bed of concrete good rubble-work is commonly built to or a little above the surface of the ground, mainly as a matter of economy. Limestone is excellent for this, and better than sandstone, although the latter can be and has been used, either of which is more economical than granite. The writer thinks it unadvisable to use either sandstone or brick under the surface of the ground unless cement mortar is used; in fact health, comfort, freedom from dampness, demand cement to be used below ground in all cases; economy alone says lime. Is it not better to be sure of the best foundation and economize in some other part of the structure? Damp houses, cracked walls, doors and windows out of plumb, attest the truth of the above; and what is more, how many walls actually fall before completion of the structure and afterwards, costing a thousand times more than was necessary to have put the foundation in properly. Sometimes timber is laid on the foundation-bed in two layers crossing each other: this is only admissible when a permanently wet stratum is reached.

35. Concrete is used in large quantities under all important structures, and especially under bridge piers, abutments, retaining-walls, etc., in masses varying in depth from 2 to 40 feet, particularly in very deep foundations, where the pneumatic caisson is used. This will be particularly alluded to when we come to discuss the subject of Pneumatic Caissons. It is also used to make enormous blocks of stone where, exposed to the action of immense moving forces, such as is in exposed conditions on the sea-coast, in constructing breakwaters, it would be very difficult if not impossible to transport blocks of the size desired. The concrete can be manufactured at points convenient to the site. Structures alluded to in the last paragraph will be discussed more in detail in another article.

36. On the foundation-bed when concrete is omitted, or on the surface of the concrete when used, what may be called the lower part of the body of the structure is constructed. This may be of brick or rubble, and in very large and important structures may be of first-class masonry, hereafter to be described in more detail. Brick is sometimes used, and is commenced with one or more footing-courses, that is, courses projecting from a quarter to almost one half the length of a brick —from 2 to 4 inches. This is not necessary when the wall springs from rock or a bed of concrete, as no spread of base of wall is necessary in this case, but is generally done. Outside bricks for projecting courses should be all headers: this is always done when the walls spring from clay or sand; then above the footing-courses the body of the wall is carried up with the prescribed thickness.

37. When this part of the work is of rubble the same rule is followed, except that the rubble wall is carried up to the surface of the ground with a little greater breadth than the body of the wall alone, so as to leave a small offset. When this part of the wall is under very large and important structures, such as bridge piers, and is made of first-class masonry, there are generally several footing-courses, forming a series of steps so arranged as to leave a small offset just under the surface of the ground or water, where the neat work commences. The different kinds of masonry will be fully described, the proper kinds of bond and material used, and all technical terms used will be explained in another article.

38. The crushing strength of concrete has never been fully determined, and in fact but few experiments have been made. Theoretically it should continue to harden indefinitely, and all that could be done would be to subject cubes or blocks to compression (noting carefully the kind and the proportions of ingredients) after the lapse of a certain time, and after intervals. This would give us the strength at that age, and by comparison the rate of increase of strength; but enough is known to establish the fact that it will in general acquire in a short time the strength of ordinary sandstone. It is claimed by some authorities that the set or hardening is delayed by pressure. For this reason it is often prescribed that each layer

shall be allowed to set before adding another or before commencing the masonry. This cannot be done in large structures on account of the delay that would be caused. The small amount of weight added each day could not cause any trouble.

ARTICLE IV.

BUILDING STONES.

39. THE most important properties of rock suitable for building purposes are the Structural and Chemical. In regard to their structural character, they are divided into the unstratified and the stratified, or those which show no distinct layers or beds and those that do show such layers or beds more or less distinctly. These properties are of great importance, as concerns both the strength, durability, and economy of structures. The unstratified rocks are generally the hardest and the strongest, and can be obtained in immensely large blocks, but at the same time are expensive to quarry and dress into proper shapes; they are compact, and have a low absorptive power; all of which renders them valuable for structures of great magnitude. Of these the most common are granite and syenite. The stratified rocks vary much in strength, durability, and compactness, and are formed in distinct layers, varying from the laminated or slaty structure in thickness, to that of several feet. The best kinds are hard and strong and durable, easily quarried, easily cut into desired shapes, and are widely distributed over the country, and consequently are our most useful and common building stones, are used in piers, retaining-walls, and walls of houses. Being found in many colors and combinations of colors, they produce a fine architectural and ornamental effect; of the most common and useful are marble, limestone, sandstone, and slate. Each of these kinds will be considered in some detail.

40. As to the chemical composition of stones, they are divided into three classes, viz., silicious, calcareous, and argil-

laceous stones, as these several substances predominate. The principal silicious stones are granite, syenite, and sandstone; of the calcareous stones, marble and compact limestone; of the argillaceous stones, clay slate.

41. Granite is unstratified and silicious, and consists of quartz, feldspar, mica, and hornblende. Its valuable properties are greater the more quartz and hornblende it contains, and less in proportion to the feldspar and mica contained; but owing to its great hardness it is seldom used, except for building lighthouses, breakwaters, and large public buildings. Owing to its great cost it is seldom used in bridge piers, unless abundant and near at hand. Granite chips badly when exposed to heat.

41½. Sandstone is stratified and silicious, and is composed of grains of sand commonly cemented together with a compound of silica, alumina, and lime. The best qualities of sandstone are those in which the amount of cement tying the material is small and composed mainly of silica, and the grains are well-defined and angular. Much cementing material, when composed largely of alumina or lime, indicates a weak sandstone, and especially if the grains are rounded. It exists in various degrees of hardness, compactness, strength, and durability; is found of almost all colors, and makes beautiful and ornamental fronts to houses; and being widely distributed, it is rendered at once the most useful and convenient of building stones. Owing to its more or less distinct stratification, its porosity, and consequently high absorptive power, it should always be placed in structures on its natural bed, so its layers may be perpendicular to the direction of the pressure; otherwise the action of frost will cause disintegration and scaling off, as well as affording less resistance to the pressure; and if much lime is present it decays rapidly when exposed on the sea-coast or to sulphurous vapors.

42. It is generally conceded that neither a physical examination nor a chemical analysis, nor even an actual specimen test for crushing strength of a fresh-quarried stone gives even an approximate idea of its suitability for building purposes;

but these combined with some other conditions, such as its appearance on exposed faces of large masses, should in general furnish satisfactory indications of its general properties. An exposed face of a mass of sandstone which we have good reason to believe has been exposed for a very long time should present the following appearance: The exposed surface should present a hard, rather dark-colored skin, of about an inch or two thick; the interior surface a little softer, and generally of a lighter color: this indicates a stone that hardens on exposure. All angle lines, vertical or horizontal, should be sharp and well defined. A rough exterior surface, with cavities of greater or less size and depth, with rounded corners or angle lines, indicates a soft variety of stone, and one that wears and disintegrates on exposure.

43. The writer examined the sandstones bordering the Ohio River for many miles east and west of Point Pleasant, W. Va., and also many miles up the Kanawha River, in order to select a quarry for stone to be used in a bridge at that place; and in this limit, although finding many kinds different in their properties, and getting all information possible from residents, he concluded that it would not be safe to risk the use of them in the large and exposed piers, and it was determined to use the Hocking Valley sandstone from a quarry over one hundred miles distant by rail: this was apparently the softest variety examined, was of a dark-brown color, and spawls could be easily broken in the hand; exposed surfaces in quarries, however, presented a good appearance. A block of sandstone could be worked when just from the quarry with an ordinary pick. Our decision, however, was based on the fact that we found dams, piers, walls of houses, built of this stone, some of which we were informed had been built forty or fifty years prior to this time, and still bore the tool-marks, and were now found to be very hard; consequently we used this stone to a very large extent.

44. We subsequently found a quarry a few miles up the Kanawha River. This stone presented a favorable appearance in the quarry, and numbers of bowlders, some very large, were

found on the hillside which seemed to be harder than the quarry stone, and showing no signs of disintegration; consequently some of the piers were built of this stone. The bowlders when large enough were freely used; the color of this stone was something like rich cream. Another stone found near this quarry, on the other side of the river, of rather a bluish color, was extremely hard in the quarry, had a high compression strength when freshly quarried, but in parts of some structures built of this stone there were plain indications of scaling and disintegration: this was used to a very limited extent, and mainly for backing stone and in concrete. These facts are mentioned to show how uncertain appearances are, as well as the actual specimen test for crushing strength, unless the stone has been quarried for some time. It is always desirable, if possible, to know that a stone has stood the test of time in actual structures; but often we have to do the best we can, guided by such tests and indications as above referred to.

45. Sandstone may be then divided into two classes: those which, though soft at first, harden on exposure; and those which disintegrate and decay on exposure, though they may be hard at first. The first alone should be used for building purposes. Sandstone stands exposure to fire better than granite.

46. The writer collected a number of samples from the different quarries examined, and from each two or more specimens were carefully dressed into exact cubes 2 inches on edge each way, and subjected them to crushing, using smoothly dressed white-pine cushions cut of exact size of the cube; these cushions, about one eighth to one quarter of an inch thick, were placed on top and bottom of cube to be tested. All the samples tested by him were strong enough to bear any reasonable pressure, varying from 3000 to 5000 lbs. per square inch, and in general even the softer specimens of sandstone will stand the pressure; but decay is the great danger to be avoided.

47. Limestones, stratified and calcareous. Marble is generally considered as a pure carbonate of lime, and is strong and durable and at the same time easily cut and dressed; and

from its variety of color in the same stone, as well as the variety of solid colors in which it is found, together with the high polish it will take, it is largely used for ornamental purposes, and also in many large public buildings as well as in private houses, but owing to its great cost it is not used in ordinary structures, and in addition it is not so widely distributed, yet marble quarries are claimed to exist in almost every State of the Union. Many limestones are susceptible of a high polish, and present a very beautiful surface, and are called marble for this reason.

48. Compact limestone is what might be called an impure limestone, containing greater or less proportions of silica, alumina, and iron, or these combined; and the qualities of the stone for building purposes depend more or less upon the amount of these foreign ingredients. But, generally speaking, any compact, hard, and fine-grained limestone is one of the most useful and common building materials. A loose, porous limestone should not be used; however, some of the soft varieties are found to harden on exposure. These stones are often difficult to quarry and dress, and often cannot be obtained in anything like regular shapes, and are therefore useless for anything but rubble work. Other varieties occur in well-defined layers of thicknesses from 1 inch to 2 feet, are easily quarried, require but little dressing, and are both economical and durable; should always be laid on their natural beds, and no excuse can exist for not doing so (in sandstones it is difficult to determine in some varieties which is the natural bed). Its absorptive power is small, and therefore it is not liable to disintegrate by action of frost. It will not stand exposure to high temperature, and is rapidly disintegrated in case of fires in cities. The pure varieties of limestone, when properly burned, yield the ordinary quicklime, and those which contain certain determined proportions of silica and alumina yield hydraulic limes. Limestones effervesce with acids—a distinguishing characteristic.

49. Argillaceous Stones. The only variety of these stones of any value to the engineer is what is known as slate. Its

principal use is for roofing houses. This is a stratified stone, and when it can be split into very thin layers it has what is said to be a laminated structure. It is found of several colors, but the darker colors in general indicate great strength and durability. It is almost impervious to water.

50. A table of the resistance to crushing of these several kinds of stone has already been given. The absorptive power of these stones can be arranged according to a descending scale as follows: Sandstone, compact limestone, marble, and granite,—the two last practically absorbing no water at all. The absorptive power can be easily determined by weighing a specimen dry, and then, after being immersed in water for a reasonable time, the increase of weight will determine the amount of water absorbed. After removing from the water, the surface water adhering should be allowed to drip off. As to resistance to heat, the order may be taken as follows: Sandstone, granite, limestone, the last being entirely decomposed under the influence of intense heat.

ARTICLE V.

QUARRYING AND STONE-CUTTING.

51. IT has been formerly stated that it is the duty of engineers to design and build structures suitable to the purpose in view, and it is easily in the recollection of the present generation when the engineer, so called, was expected to know how to do almost everything in the way of utilizing and controlling the forces and materials of nature, in promoting the comfort, happiness, and prosperity of mankind; and as at the period referred to but little was known, it was possible for one man to know and to put into practice what was known,—mainly by a sort of rule-of-thumb method. This perhaps may have originated the prefix Civil to the general term engineer. But in the past few years such development and progress has been made in the sciences and arts, that it has become necessary to divide the subject into almost numberless branches, all more or less allied and interlinked, but each so broad and deep that he is fortunate who has the time to master any one of its subdivisions; and here we have the civil, the mechanical, the hydraulic, the city, and now the electrical engineer, to say nothing of the architect and the bridge engineer. Bridge construction has now become an almost exclusive science. Consequently it is difficult to know how much of each of these any one should know, in order to claim or deserve either of the above titles, and equally difficult to determine the border-line between any two of them.

52. These considerations must be the writer's excuse for introducing several subjects that would seem to have not the least connection with what he shall give as a title to this volume, namely, a Treatise on Foundations, and must at the same time explain the omission of many things that should be included.

QUARRYING.

53. Quarrying is purely an art, and little can be learned of it except by experience. The illiterate quarryman will take out more stone, and in better shape, in twenty-four hours, than the ordinary engineer will do in a month; but still it seems that he should at least have the benefit of the few general principles that are known. All stones, even the granite, have, to the expert, well-marked division lines; limestone and sandstone have them generally well defined, and the first principle in quarrying should be to detect these division lines, not only as a matter of economy, but also to obtain the blocks of the proper size and shape. Another principle is either to use no powder or very little explosive material, except in case of the very hardest kind of rocks, such as granite, and then with great care and judgment, as it is hard to determine the effect of an explosion upon the portions of the mass loosened, it may produce injurious effects, which may remain unseen and seriously impair the ultimate strength and durability of the material. However, blasting with powder or dynamite is usually resorted to, the large volumes loosened and time saved compensating for the waste caused by the explosion, and in addition

a judicious use of small charges seem to produce better results than larger charges. Limestone in layers can generally be quarried by the use of picks, crowbars, hammers, and wedges. Sandstone can often be readily quarried by the same tools, aided by the use of the plug and feathers, which consist of a small steel wedge and two iron semi-cylindrical pieces ; but unless the stratification is well defined, blasting is resorted to; and often it is found advantageous to throw down very large blocks of the material, and subsequently subdivide these, either by small blasts, or by the use of the above tools. The plugs and feathers are used by first drilling a series of small holes a few inches deep in a line, then placing two feathers in each hole and driving the plugs between them. No attempt is made to drive each plug or wedge any great depth at any one time, but a blow of a hammer is given in succession on each plug in the line, and the stone will soon split entirely through the block along the line of the holes.

54. When blasting is necessary, holes have to be drilled of greater or less depth, and varying in diameter from 1½ to 2½ inches. These holes are then partly filled with a large-grained powder or dynamite, and exploded either by ordinary fuse or electricity; several, at distances apart depending upon circumstances, are fired simultaneously, and at definite times, such as at noon and at the end of the day, when the men can be away at meals, in order to have plenty of work ready when they return. There seems to be no fixed rule as to amount of explosive material used, as conditions vary greatly, even in the same quarry, and nothing but experience and good judgment can be depended upon; an ordinary rule is to fill the hole about one-third full of powder. The hole should then be filled by first placing a few inches of dry clay on top of the powder. This clay should be free from sand or grit, and should be gently tamped or compacted with a wooden rammer, to avoid premature explosion. The hole can then be filled with sand or other rubbish. Results seem to show that from ½ to 2½ pounds of powder are required to loosen thoroughly a cubic yard of rock in place. As

generally stated, the mass of rock loosened bears some proportion to the line of least resistance cubed, and estimated at about twice that result, it being understood that that line is the shortest distance to the exposed face of the rock from the charge; but this is often far from the fact, as this least resistance depends upon the nature and character of the material, the position and direction of the hole and the manner of tamping or filling the hole.

55. The holes can be drilled or bored by hand or machinery. There are three methods by hand. In the first, a long iron rod, with a steel chisel-shaped cutting edge, is lifted by one or two men to a certain height and then allowed to drop in the hole, giving a slight turn after each blow. In the second, an iron rod of varying lengths, according to the depth of the hole required, is held by one man, and two men strike on the top of the drill alternately, the man holding the drill turning it continuously as the blows are struck. The first of these is considered more efficient. In the third, known in practice as "ball drilling," one man has a long iron rod, with a specially made point, this rod he simply lifts and throws into the hole, as it were. The accuracy with which they handle the drill and the rapidity of the work are certainly astonishing, and perhaps the reason that it is so seldom resorted to is that it requires the skill of a drum major to keep the hole straight and hit in it every time. A day's work in drilling will vary from 5 to 15 feet per man.

56. Machine-drilling is on the same general principles, except the power is applied by steam. The drills are moved forward by blows or turning, or both, and of course on extensive works progress is more rapid and economical than by hand-drilling. The diamond drill is in very common use, is expensive in its first cost, and rarely used when limited quantities of material are to be quarried. The tube or drill in this case is a pipe or hollow tube, having a head at the bottom, in which is placed number of small black diamonds, projecting slightly from the surface. This is caused to revolve rapidly and cuts a cylindrical hole. The material, in the form of dust or small particles of the stone, is brought to the surface by forcing water down the tube through holes in the head and returning through other channels on the outside of the drill. In hand-drilling the débris or dust is removed in a very crude way—by first removing the drill from the hole and inserting a long branch of some kind of wood, split and broomed at the end, or by the use of small wooden or iron spoons, connected to the end of a pole. During the process of drilling, water has to be continually poured into the hole. It aids the drilling by softening to some extent the material, and keeps the end of the drill cool.

57. Dynamite has many times the explosive power of powder, varying according to the percentage of nitro-glycerine it contains, and is generally used in place of powder when a violent and sudden explosion is required, as blasting in railroad cuts or in removing large masses, regardless of the shape or size in which they are thrown down; but in quarrying for dimension stone great care should be used to avoid too much shattering of the stone and breaking into small pieces. Dynamite generally is sold in candles, so called, of almost any diameter and length, and containing different quantities by weight, wrapped in brown paper, which makes them convenient to handle, and apparently no more dangerous than powder, as certainly the men handle it as carelessly as they do the ordinary blasting-powder.

58. Quarries should always, when practicable, be opened on hillsides, so as to obtain a large vertical working face, and the top soil stripped off until a solid ledge is reached over a considerable area. This stripping is generally expensive in first cost. This stripping can be done economically and rapidly with a water jet where water in sufficient quantities is convenient, but the necessary machinery is expensive.

59. The most economical condition for quarrying is when all of the stone, both large and small, can be utilized, as otherwise the waste will be very great. All things considered, the cost of the construction will largely depend on this, as in order to get dimension or face stone for piers there will necessarily be a large quantity of large stone unfit for face stone, and at the

same time a large quantity of small stone, such as one-man stone, and spawls suitable for breaking into stones for concrete. Consequently, if a series of bridge piers can be so planned as to combine in the same pier all of these sizes and shapes, the cost of construction would evidently be lessened. In many cases this can be done consistently with the recognized and good practice, the broken stone and one-man stone used under ground and under water, and the large, rough stone used for backing in the piers: or in some of the piers the backing could be large stone and in others concrete; or even a combination of these in the same pier, alternating the courses, one course backed with large stone and another backed with concrete, the latter producing seemingly a stronger pier than that built by either of the other methods. Absolute uniformity is the common practice, and dependent, as has been stated, practically, on the whim of the chief engineer. Surely common-sense would justify the combination pier, with knowledge before us that either independently has been used repeatedly and with satisfactory results. (See Figs. 18 and 19.) Some engineers will not allow the backing stone to be of a different kind from the face stone, when either are recognized as good enough for the entire structure. One reason assigned is that different kinds of stone have different degrees of expansion and contraction under changes in temperature. Probably the greatest differences in hardness and strength exist in granite and sandstone. According to Rankine, granite expands .0009 of its length in a change of temperature of 180° Fahr., and sandstone varies in the same range from .0009 to .0012 of its length. Or take 90° as probably the greatest possible range of temperature likely to occur, and we have for extreme differences .00045 and 0006. But this range of temperature in a mass of masonry is improbable, and the fact is that the expansion is microscopic.

60. Many engineers put upon themselves onerous and often useless, if not harmful, duties, such as specifying for each pier of a bridge the exact size of each stone in a pier and in each course. This necessarily leads to delay, confusion, and expense.

A good quarry foreman always makes a diagram of each course in a pier, and can easily select from the supply in the yard such stones as will fulfil the conditions of good masonry, which are marked and forwarded, together with the diagram, to the site of the work, and only occasionally requiring any cutting, except for a closure, unless in case of rejection of the stone when delivered. These lengths and sizes may vary slightly from any arrangement that would be made by the engineer, but in ordinary massive masonry would present as good an appearance and have equal bond. The proper rule is to fix definitely your limits upon the sizes, extent of bond, proportions of headers and stretchers, and allow reasonable variations between them. Harmony will prevail, good work be secured, and money be saved. Onerous requirements, especially when evidently useless, produce often the exactly opposite result.

61. Almost all large and important works are done by contract, for the obvious reason that, all things considered, they can be done more cheaply and more expeditiously in this way; and although the writer has met with rascals in almost all departments of the contracting business, he is glad to say that he is not one of those who think that all or even a large majority of them can be considered as belonging to that class. On the contrary, he believes that they are otherwise; and he would rather have a reliable contracting firm to do work without close inspection, if the firm has confidence in his justice and good judgment, with reasonable requirements, than to conduct the work in accordance with the most onerous requirements and most rigid and ruthless inspection without such confidence.

ART. VI.

STEREOTOMY.

62. Stereotomy, as the art of stone-cutting is called, is an important and interesting subject, as well as a difficult one in practice; but it rather belongs to the domain of the architect than that of the engineer, and except in the ornamental parts

of structures, the forms used are simple and we may say, few in number. In ordinary and massive structures the surfaces are plane or cylindrical, circular or elliptic, and all the stones in the same structure are generally of the same shape where plane surfaces are departed from. The true "skew arch" is an exception, every stone having a different shape and size, and of several kinds of curved surfaces; but as commonly built the surfaces are either plane or cylindrical. The more ornamental parts of a structure require a profound knowledge of forms and combinations of forms, geometrical shapes and lines and the manner of constructing them on paper, templets and models; the skilled stone-cutter does the balance, and for the simple forms the more intelligent of these can do it all with a little aid in calculating the radii necessary. In ordinary structures all the stones have plane surfaces and the angles are right angles. In piers in rivers the ends are sometimes cylindrical, circular, or elliptical or wedge-shaped, and always when exposed to heavy flows of drift or ice. In arches the sides are plain. The bottom is a part of the surface of a cylinder; the top is generally left rough. The whole stone is the frustum of a wedge, the sides being slightly inclined, so as to conform to the direction of the radii of the arch. Ordinary templets or models used in cutting the surfaces of the stone are made of wood. (Figs. 3 and 4.)

63. The tools used by the stone-cutter are hammers of various sizes and weights—both ends blunt, or one end chisel-shaped or pointed, or both ends chisel-shaped or pointed, and some patent hammers, and in addition tools called the point and chisel. Stone-cutters generally provide their own tools.

64. The work in general is performed by first cutting chisel-drafts around the edges of the stone about 1½ inches wide—these all in the same plane; and by the aid of a straight-edge, a piece of timber about 6 feet long, 3 inches wide, and 1 inch thick, the enclosed rough stone is dressed down to the same plane. For a curved surface two chisel-drafts are cut, one at each end of the stone, to conform to the templet, and the intermediate rough stone cut out in the same manner. Intermediate chisel-drafts are cut if the stones are very large. Stone-cutters are generally charged with the loss, if by carelessness they ruin stone in cutting.

65. It is generally prescribed that the beds and joints shall be plane surfaces at right angles with each other. This is seldom fully realized in practice, and a very slight concavity is in some respects favorable. The straight-edge should be applied longitudinally, transversely, and diagonally to see that the stone is out of wind or not warped, and the surface of the stone should closely conform to the straight-edge. It is also advisable to dress the side joints a little slack; that is, if stones 2 feet broad on bed are placed touching on the face, they should be open from $\frac{1}{4}$ to $\frac{3}{4}$ inch at the back: this favors filling the joints easily. Stones are not required to be absolutely of the same width from face to back, and for the entire depth of the stone; this would be called close dimension stone, but it is generally specified that they shall be of the same width for 1 ft. to $1\frac{1}{2}$ ft. from the face. The bottom bed of a stone should be cut strictly to the same plane over its entire surface; the top bed may have slight inequalities on its surface, as they will be necessarily filled with mortar, and it is generally allowed that the large backing stones may have from $\frac{1}{2}$ to 1 inch less thickness than the face stones, but should in general have almost as good beds.

66. Sometimes a chisel-draft is required to be cut around the edges of the stones, to enable the mason to set the stones exactly over each other. A good clean-cut, straight pitch-line will answer fully for this purpose and cost less, but it is advisable generally to cut this draft at the angles or corners of piers; but this is not always done. The writer always determined the exact centre and laid off the masonry to calculated dimensions every fourth or fifth course, so as to avoid any possibility of the pier getting out of plumb.

67. Stone-cutters are very apt to cut the stone so that it will not be as thick on the back as it is on the face. This should not be allowed, as it makes the mortar joint too thick

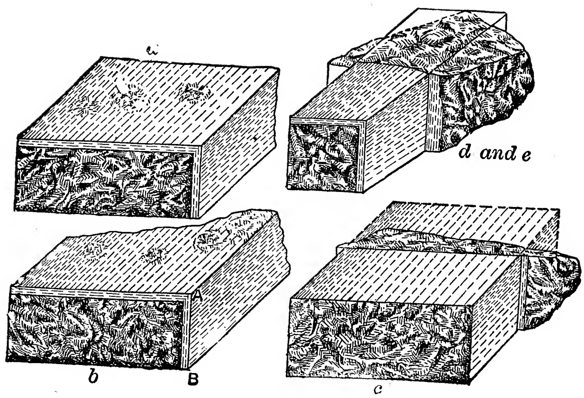

FIG. 3.

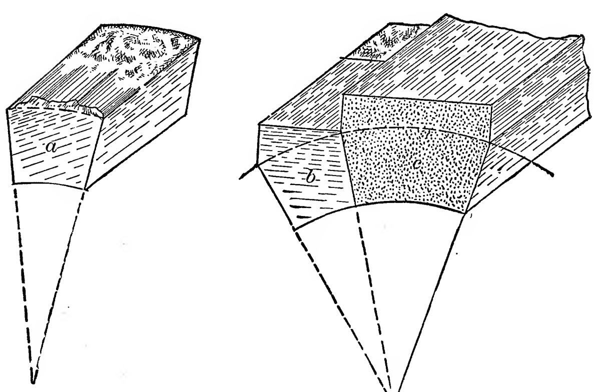

FIG. 4.—VOUSSOIRS.

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under the stone. This should be carefully measured with a rule, or better with an instrument made as follows (Fig. 5): A batten 3 or 4 feet long with a projecting piece at the bottom, and a sliding piece attached; the projecting piece is placed under and against the stone; the sliding piece is then lowered to touch the stone on top and fastened; this scale is then applied to several points front and back, which will readily show any variation in the thickness. The face stones in each course should have absolutely the same thickness or rise of the course. In most massive structures the face of the stone is generally left rough or rock face, and generally the extent of the projections is immaterial, but it is usual to limit it to 4

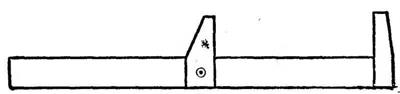

FIG. 5.—GAUGE FOR SIZING STONE.

or 5 inches; but the ends of piers below high-water and for some distance above, where there is much drift or ice, should be dressed to a reasonably smooth surface, or even bush-hammered,—that is, dressed as smooth as possible,—and this should extend below the water. The stones are all cut to the proper batter in the yards, except for the stones of the footing-courses. The stones for each pier are generally cut in advance of the building, and piled up at some convenient place, arranged according to courses as far as practicable, increasing in thickness of courses from the bottom to the top,—the inverse order from that in which they are to be used in the structure,—in order to avoid too much labor in handling.

ARTICLE VII.

MASONRY.

68. IT will be best to adhere strictly to the common classifications, as generally understood in this and other countries. We shall, however, reverse the general order and commence with the inferior kind, as follows: Dry stone walls, ordinary rough rubble walls, rubble walls in course, block-in-course masonry, ashlar masonry. There are also some combinations of these, as walls made with ashlar or block-in-course or brick, and backed up with rubble. The most usual and widely distributed stones for building purposes are granite, marble, limestone, sandstone, and brick.

69. Granite, owing to its extreme hardness, is seldom used except in the most important structures, such as lighthouses, large piers, and public buildings, when cost of construction is not considered. Marble, though not so hard, and can easily be worked into ornamental shapes, is likewise only used in buildings where the cost is ignored. Therefore for ordinary purposes we are compelled to rely upon the following stones.

70. Limestone is one of the most useful, most generally used and widely distributed of the building materials, and can generally be relied upon. It is found in various conditions of stratification, from the gnarled and twisted to that of the most perfect layers, in various thicknesses from a few inches to two or more feet. In this condition it is easily quarried, comes out with good beds, requiring little or no labor in dressing and cutting, and can be gotten of almost any length and breadth. Its strength and durability depends upon its compactness. It will not stand a high heat, under which it disintegrates, and also when exposed to an acid atmosphere.

71. Sandstone is also widely distributed, strong and durable, and can easily be cut, sawed, and dressed; occurs in thick strata, and can easily be quarried in blocks of almost any dimensions, all of which conditions render it a useful and valuable building material for almost any kind of structures, but withal one of the most uncertain and treacherous of stones, as it exists in all conditions of compactness and hardness; but unfortunately the hardest varieties when first quarried may be the least durable, and some of the softest varieties, which can be dressed with a pick when first quarried, prove ultimately the most durable. Those varieties which present sharp grains with a small amount of cementing material are generally the best. The safest plan, however, is to examine structures, chimneys, steps,

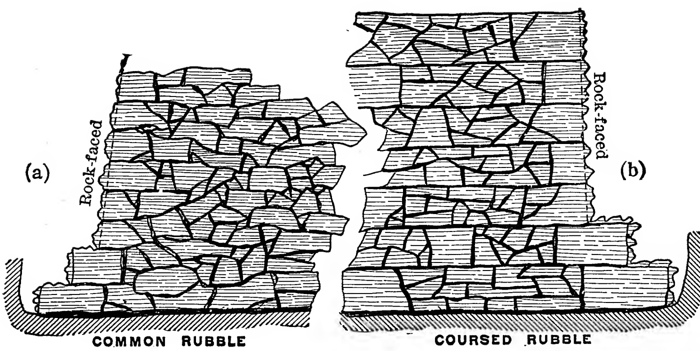

FIG. 6.

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etc., built of this material and known to have stood for a long period of time. These can generally be found, but in the absence of this guide we have to do the best we can. Sandstone is porous, and special care should be taken to build it on its natural bed, but in many varieties of sandstone it is hard to determine the direction of the stratification. Mineralogy will give the color, general appearance, and locality where found, and other general properties. Chemistry will enable the reader to determine the exact composition, and engineers should be reasonably familiar with these subjects.

72. Dry stone walls, although not capable of bearing any great weight, unless constructed of regular-shaped stone, with good beds, are useful for retaining-walls of small height, and can be built of almost any shape and size of stone, and even of round river jacks or bowlders, and answer well in those cases where no danger or risk could occur if they did fall down, and where great economy is desired.

73. Rough rubble masonry is built of any shaped stones, just as they may come from the quarry, without hammering or any kind of dressing; but generally one or two man stones down to the smaller spawls are laid without regard to continuous horizontal joints or beds, but with special care to breaking joints vertically, by overlapping the stones, producing what is called "bond," and well bedded in mortar, generally of common lime and sand; vertical joints are also filled with mortar, and any openings between the larger stones on the beds or joints should be filled with smaller stones bedded in mortar. Thus built, it will make a wall of considerable strength, especially if built with cement mortar, and in the latter case will make a good arch ring for small arches, its strength somewhat exceeding the strength of the mortar used when hardened; and when faced with a good coating of stucco or cement mortar, can be made to present a neat face. This kind of work is used in the lower part of foundations to carry even very heavy loads, and is suitable for ordinary retaining-walls, and for many purposes where economy is a matter of importance. (Fig. 6, (*a*).)

74. Rubble walls in courses. In this class of work there

are no regular courses of uniform thickness, the joints between the stones, both in vertical and horizontal planes, being broken. The side joints need not be vertical, and the stones may be only hammer-dressed on joints and bed; but with good mortar and reasonable care in building so as to have a good bond, this class of work can be made strong and durable, and, where looks are not considered, would answer almost any ordinary requirement, and may be made to harmonize pleasantly with rustic surroundings, and possesses one important element—economy. To a large extent the sizes of the stones used are unimportant, from very large to very small. It is the kind of masonry almost exclusively used for backing retaining-walls. (Fig. 6 (*b*).)

75. A better class of this kind of work, in which the beds and joints are dressed, makes a strong and durable structure. The irregularity in the size and shape of the stones, provided the joints between the stones are broken horizontally and vertically, the rough undressed face of the stone, all combine to produce a fine architectural effect; some of the handsomest churches and other structures are built of this class of masonry, though hardly to be recommended for heavy structures or structures subjected to forces tending to drag or knock the smaller stones out of place, such as bridge piers, which have to stand blows and shocks from driftwood, ice, etc., will form nevertheless a substantial and economical structure for ordinary purposes.

76. To build this class of work great care must be taken to secure good bond, both longitudinally and transversely, and due care should be given to proper adjustment and distribution, over the entire surface, of the larger and smaller stones.

77. Block-in-course work. This class of work varies from the above in having regular courses of uniform thickness, varying from six to ten inches. The stones are cut into regular blocks of prescribed length and breadth, the length about three times the thickness and the breadth from one to two times the thickness, beds and joints cut true and at right angles to each other. About one fourth of the faces should show headers,—that is, stones whose ends show on the face of the wall and extend

at least three times the depth of the course into the wall, the breadth of the headers being at least equal to the thickness of the course,—and they should rest on the stretchers below as nearly over the centre as possible, so as to allow for overlap or bond of at least one third of the length of the stretcher, the stretcher being a stone the length of which is shown on the face of the wall. Sometimes stones are found in strata of the thickness requisite for this kind of work, are easily quarried, do not require an excessive amount of cutting and dressing, and consequently are well adapted to the purpose. Sandstone and granite are generally quarried in much thicker blocks, and are therefore better suited to structures requiring thick courses, and can be more economically used in the larger blocks. This class of work is suitable for almost any structure, unless exposed to some kind of shock, as in case of lighthouses and bridge piers, and presents a neat appearance, but is not economical unless the stone comes in the quarry in small blocks and with good natural beds.

78. Ashlar Masonry. This class of masonry stands at the head of the list, and is used in all important structures, such as large piers for bridges, lighthouses, breakwaters, and large public and even private buildings. Granite is used for the most important structures regardless of cost, but limestone or sandstone are used when cost enters as an important factor. The strength of this class of masonry arises from the large size of the blocks used, the care taken in cutting and dressing the stone, the care taken in building the structure, and the extent of the bond obtainable, both longitudinally and transversely. It is laid in regular courses, of thicknesses varying from 1 to 3 feet. The length of stones from 1 to 4 times the thickness and breadth from 1 to 2 times the thickness, and with a bond from 1 to $1\frac{1}{2}$ times the thickness. The side and bed joints are dressed to plane surfaces and at right angles to each other; it is not desirable that these should be perfectly smooth surfaces, but should present a series of shallow ridges and hollows, such as would naturally result from finishing with a pointing tool. They should be nearly true throughout the surface to

a straight-edge, rather concave than convex towards the centre of the surface. There is little danger of stone-cutters leaving the stone convex on the surface, as it is difficult to set such a stone, and tends to leave large open joints on the face. The danger, however, is of cutting the face concave, thereby insuring a thin and neat joint on the face. The danger here is of throwing the pressure on the edges of the stone, causing the edges to chip and spawl off, thereby defacing the face of the work. If resulting in no other harm, this effect can be seen on the face of many structures.

79. Ashlar masonry, however, in large piers is only used on the two faces and two ends, leaving a hollow centre space; this must be filled up with something. This filling, whatever it is, is called "backing," and depends to a large extent on the whim of the chief engineer. Some engineers say ordinary rubble is good enough, some say concrete; some say large stones of the same thickness as the face stone, only leaving small space of from 6 to 12 inches to be filled with rubble or spawls. Few however, require the backing stones to be dressed as closely as the face stones, but they should be brought to a good average even surface on the beds, though some require the backing stones to be dressed as true as the face stones. This latter may be best, but if the other is good enough, why go to the greatly increased cost. The only reason that we can build ashlar masonry at the prices now existing is based upon the rough backing being used, as the profits are drawn almost entirely from this source. (Figs. 18 and 19.)

80. The joints in ashlar masonry to be filled with mortar vary from $\frac{1}{8}$ to $\frac{1}{2}$ inch in thickness on the face. In actual practice, except in some special cases, the larger limit is probably reached in most cases; there is no need of exceeding this limit.

81. Assuming that the face stones have been laid with the proper proportions of headers and stretchers, how shall the enclosed space be filled? The writer would fill with large backing stones of the same thickness as the face stones, filling the small vacant spaces with spawls. A bad habit of masons in this filling is to put down a pile of small stones, then smear a little mortar over the top. This should not be allowed. A thick bed of mortar should first be thrown in, and the small stones pressed and rammed into the mortar, then another layer of mortar and stones pressed in, and so on. This insures solid work, and is as easily done, if not more so than the other. The spaces need not exceed 6 inches on an average. The backing stone should be laid so as to break joints both longitudinally and transversely. (See plan of pier, Fig. 18.)

82. The practice with some engineers, after laying the large backing stone in place, taking care in all cases to break the joints in both directions, so as to bond the entire wall both longitudinally and transversely, is to fill the vacant spaces with broken stone of varying sizes, and then "grout" the work, that is, pour liquid mortar into these places until they are filled, first pouring in a liberal quantity of water; when filled with mortar the water will rise to the surface. The trouble is that under these conditions the cement and sand will to a large extent separate, the cement rising to the top, thus forming a series of layers of sand with little cement and of cement with little sand, as the sand will invariably sink to the bottom. This at least is the writer's experience. Others claim that it is best and insures a solid wall. It is largely practiced.

83. The second-best method is to fill the entire space between the face stones with good concrete, with headers reaching well back into the wall and some backing stone overlapping the tails of the headers from opposite faces. It has always been a puzzle to the writer why this plan is not more generally followed: it would certainly insure a solid strong wall, is more rapidly put in and probably more economical than the first plan, but some prejudice exists against it. (See left half of plan, Fig. 19.)

84. Lastly, to fill the vacant space between the face stones with rubble. This can be done either by carefully bedding the larger stones in mortar, and filling in between these with smaller stones and spawls well pressed in the mortar, or by simply throwing large and small stones in the vacant space

until filled, then pouring grout over the entire space until all interstices are filled with mortar, as above directed. This method is doubtless less costly than either of the other two. It may be good enough, but for some, no doubt good, reasons is rarely adopted for important works. (See right-hand half of Fig. 19.)

85. In whatever manner the backing is constructed, the wall of the pier is carried up from course to course, each course being entirely completed before beginning another course, as it is a bad plan to build a part of several courses and leave a series of steps, and then build up the rest of the pier bonding on the older work, which can rarely be done as well as in completing entirely each course.

86. The neat work commences at or a little below the surface of the ground or water, on top of the footing-courses which was called the foundation, and generally diminishes in size gradually to the top of the wall. This gradual decrease in length and thickness is called the batter, and is generally at the rate of $\frac{1}{2}$ inch to the foot all round, or in other words diminishes in length and breadth 1 inch for each vertical foot from bottom to top. The bottom dimensions are determined from the top dimensions, which are fixed according to the purpose for which the structure is intended. In case of piers this is fixed by the bridge companies who build the iron work or superstructure, and adding 1 inch for each vertical foot of height gives the dimensions for the neat work at the bottom. The spread of the footing-courses is determined arbitrarily, but generally arranged so as to give from 2 ft. to one half of the bottom width of the neat work on each side, the projection of each course generally being from 6 in. to 9 in., or even 12 in. The footing-courses generally increase downwards by offsets or steps.

87. The appearance of the stone on the face of the work has nothing to do with the classification of the masonry, this depending entirely upon the size and shape and the manner of dressing the beds and the joints of the stones. As to the appearance on the face, whether dressed smooth, as in the

finest of masonry, such as large public buildings, lighthouses, etc., or with-chisel drafts from 1 to $1\frac{1}{2}$ inches cut all round the edges of the stone, the remaining portion being left with quarry or rock faces, or whether a simple pitch-line is cut around the edges of the stones, that is, simply cut to a sharp, straight, well-defined line, and the entire face left rough, except that projections over 4 or 5 inches are knocked off,—none of these conditions affect the strength or durability of the structure. The chisel-draft aids in setting the stones true, the one above the other, so as to avoid slight projections, and enables the mason to keep a regular and uniform batter.

88. A good pitch-line fully meets these conditions. It is considerably more economical, and in large masses of masonry permits a better and more appropriate appearance than the two first methods. For architectural effect, as well as to prevent a continuous flow of rain-water down the face of the pier, at some suitable point a string-course is built in the wall; this consists of broad stones well bonded into the wall and projecting from 6 to 9 inches from it all around, with a wash cut on the projecting portion, that is, cut on a gentle slope downwards. At the top of the wall is placed a course of large stones projecting from 6 to 9 inches all round the wall; a wash is also cut on the projecting portion: this is called the coping, the object of which is to give a neat finish to the top of the pier, to protect the smaller stones and rougher work below, and at the same time to distribute over a large surface the heavy concentrated weight above. These stones are dressed perfectly true and square on all sides, and laid with as close joints as practicable, these joints being entirely filled with a thin grout. The stones, owing to their exposed position, are generally fastened to each other by iron cramps, or fastened to the masonry below by long iron bolts, placed in holes drilled for the purpose and fastened in place by pouring in the holes after the bolt is in place either melted sulphur, melted lead, or cement grout. On top of this coping another coping-course is sometimes laid, and then large thick stones of some hard material are placed (that is, in case of bridge piers), from each of which springs an end post of the bridge with its pressure concentrated on this stone. This large stone is called the bridge seat or raising stone, and distributes the pressure over three or four coping-stones below, but otherwise is simply a matter of convenience, and is often entirely omitted.

89. The appearance of a wall of masonry, on its face, does not necessarily determine the character of the masonry. It may look well, and seemingly in accordance with the specifications; the headers may only be blocks, or "bobtails," as they are called; stretchers may have less breadth than thickness, and the interior bonds may be poor; that this not only may be the case and often is, cannot be doubted or denied. The interior condition is only fully known by the builder, the most rigid inspector cannot ordinarily prevent it, but mutual confidence and reasonableness between the two will largely do so. It is not unusual to specify that the headers and stretchers should not be less than 3 feet long, and likewise that they should not be more than 6 feet long. As to the length of the headers, it would seem better to proportion this to the thickness of the wall at that point. Walls are generally built in courses of varying thickness, and generally decreasing from bottom to top, the thicker courses being at the bottom, and the width of the piers varies from 15 feet to 20 feet at bottom to 6 feet to 12 feet at top; and with the limitation that a header should never be less than 3 feet long, the headers should generally vary, from 6 feet at bottom to 3 feet at top of the wall. A 3-foot header in a course from 2 to 3 feet thick would practically be of no use, but in a high pier it would be difficult to build it without securing a good bond throughout.

ARTICLE VIII.

ORNAMENTATION.

90. Although ornamentation is of secondary consideration in large massive structures such as bridge piers, yet a good effect can be produced by a simple string or belt course at some suitable point in its height. This is, however, seldom used with square ended piers, but with rounded or pointed ends it is usual. The curved or pointed end is generally built to a point a little above high-water, and the upper part is completed to the top of the pier with square ends, which is then finished off with a suitable coping. The string or projecting course is usually placed at the dividing line between the rounded and square end of the pier, and a low conical-shaped finish on top of the belt-course makes this passing from the one to the other pleasing to the eye. To make the templets, a platform of wood is made, a centre point fixed; a round iron pin is then driven at that point, and a straight-edge laid flat, with a small hole near one end, can be made to revolve around this as a centre: another hole is bored at a distance from the first equal to one half the width of the pier at the bottom, and a spike or pencil fastened in this will describe a proper circumference on the platform, from which the templets can be cut. A pencil in other holes $\frac{1}{2}$, 1, or $1\frac{1}{2}$ inches from each other, according to the batter and the thickness of the course, will describe the proper circle for the different courses. (For circular ends see Fig. 18.)

91. For elliptical ends they may either be a part of one ellipse whose conjugate axis is the width of the pier, or some portion of the semi-ellipse, the double ordinate or base of which is the width of the pier, in which case the foci are marked on the board at a distance apart to be determined by the shape of the point and the length of the rounded end required, which will be largely a matter of taste. At the foci drive spikes, and with a string equal in length to the transverse diameter of the ellipse, its ends fastened to the spikes, then with a spike or pencil, drawing the string tight and keeping it taut all the time, the pencil will describe the ellipse; selecting a point on the curve, whose double ordinate is equal to the width of the pier, this will be the base of the templet, which then must be cut to conform to the curve of the vertex of the ellipse. Sometimes the ends are formed by parts of two intersecting ellipses, which must be similarly constructed on the platform. The sizes of these, as said above, are mere matters of taste: the length from the body of the pier to the point is generally about equal to the one-half width of the pier. For triangular ends, the sides are plane, the base is equal to the width of the pier, and the altitude equal to one half that width. (For elliptical and triangular ends see Fig. 19.)

92. All of these ends, called starlings or cutwaters, are dressed on the exposed surfaces either smooth or approximately so, and are generally carried up with the regular batter of $\frac{1}{2}$ inch to the vertical foot, and are placed generally at both ends of the pier for symmetry, but they are only necessary at the up-stream end. These portions of the pier are not considered as bearing any part of the weight of the structure, but to split and turn aside drift and ice, or in some cases to prevent any scouring tendency by offering less resistance to the current. They should, however, be carefully bonded into the pier, and in some cases it is best to bolt them to each other. In some cases looks are thrown aside, and a well-defined cutwater is placed at the up-stream end of the pier, the lower being square.

93. A strictly called cut-water is built on the up-stream end alone. This is used where the piers are very high and thick, and large masses of ice have to be dealt with. This is built from a little distance below low-water to a point a little above high-water—generally not over 12 or 20 feet. It may be described as an oblique pyramid projecting from the body of the pier, the up-stream edge sloping towards the pier at an angle of forty-five degrees. Near this edge the sides are dressed smooth, forming a sloping prism, whose base is a triangle, the base of which triangle is the width of the pier and the altitude one half to one time that width. The remaining portion of the cutwater is solid masonry, of the same width of the pier; the end stones should be thoroughly fastened to each other by iron bolts and cramps. This form of starling will split and break immense sheets of ice of great thickness. (Figs. 49 and 49 (*a*)).

ART. IX.

ICE AND WIND PRESSURE.

94. IN piers of bridges, under normal conditions, the pressures are vertical, and as the centre of pressure is in the centre of the figure of the base, the pressures are uniformly distributed; hence there is no danger of sliding, as the bed joints are horizontal or perpendicular to the pressure, and no danger of overturning, as the pressures are all vertical, and the piers only have to be strong enough to resist crushing. But under some circumstances they are subjected to unusual forces, such as high winds, which not only act directly against the pier, but upon the superstructure and upon the train that may be passing over the bridge; also from the current acting upon large fields of ice, which sometimes gorge or bank up to the depth of many feet; and when a solid mass bridging the river exists, each pier is supposed to carry a pressure due to a mass the depth of the ice by length of a half span on either side, and from the wind pressure that is exerted on the truss and train for the length of a half-span on either side. Both of these pressures are unknown, but by assuming values for these based upon such data as we have, the problem is a very simple one. Trautwine states that the pressure per square foot exerted by the wind upon a surface exposed at right angles to its direction is equal to the square of the velocity in miles per hour multiplied by the area of the surface and divided by 200, viz., $V^2 A \div 200 =$ pressure in pounds per square foot, at 40 miles per hour, and $A = 1$ sq. ft., the pressure per square foot is equal to 8 lbs.; and for $V = 100$ miles per hour, the pressure per square foot equal to 50 lbs., and so on. A velocity of 100 miles per hour is a hurricane. The pressure from the field of ice or gorge is certainly unknown. The ice in the Susquehanna River at Havre de Grace often freezes to the thickness of 2 feet. The writer has seen it from 15 to 20 inches thick in a solid sheet from shore to shore. It moves in this solid mass 6

or 10 feet at a time, and repeadedly, before it breaks up. The cutwaters on these piers would split it from 50 to 100 feet above the pier, the mass rising up over the cutwater; and while this was going on the broken ice from a distance above would be rising on top and sinking under this immense sheet of ice to unknown depths. These facts are mentioned to show the enormous and unknown pressure to which these piers are subjected nearly every winter. This probably represents as great a pressure from this source as is likely to occur anywhere, and as showing that piers as built from necessity are sufficiently large and heavy to resist all these outside forces. Some authorities give about double the pressure from the wind as above given, but by assuming 50 lbs. per square foot of exposed surface it is doubtless on the safe side. Another formula is $P = 0.004V^2A$.

95. As bridge trusses are open work, it is generally assumed that the exposed surface is double the area of one truss for an unloaded truss, and on a loaded structure 30 lbs. wind pressure per square foot of total truss surface, and in addition an equal amount per square foot of train surface, the latter treated as a moving load; and as the good practice, though far from uniform, we may take truss and train as exposing together 20 square feet per foot of length, equivalent to 600 lbs. pressure per foot of length, and for a pier carrying two 525-ft. spans, a total pressure of 315,000 lbs., equal to 160 tons. Assuming that the pier splits the ice for a distance of 50 feet above the pier, the ice being 2 feet thick, and assuming the resistance to be 10 tons per square foot, we would have $50 \times 2 \times 10 = 1000$ tons. Moment of overturning due to wind pressure = 160 tons multiplied by lever arm (height of pier plus one half of truss, equal to 100 plus 30 equal to 130 feet) = to 20,800 ft.-tons. The ice-pressure at the Susquehanna was doubtless the greatest at a rather low stage of water, but for safety we can assume the lever arm to be 20 feet. We have 1000×20 equal to 20,000 ft.-tons; but double this and make it 40,000 ft.-tons, there results then total overturning moment equal to 60,800 ft.-tons. The weight of one

of the piers would equal 4350 tons, and the weight of one half span on either side, or one entire span, say 700 tons, and weight of empty train and cars, 200 tons, or total 5250 tons, which multiplied by one half the length of pier, equal to 20 ft., then the moment of resistance to overturning, would be 105,000-ft.-tons, or a factor-of-safety of about $1\frac{3}{4}$. These results are based upon the most unparalleled conditions, by increasing the tendency to overturn far beyond that which is likely to arise, as probably extreme pressures are assumed, and these supposed to act together, which would rarely occur, and the train is supposed empty in addition. The increased crushing pressure is not worth considering. The stability of pivot piers is certainly equal to if not greater than that of the corresponding rest piers, and in addition they are protected from ice pressure by guard piers especially constructed and entirely separate from the pier itself, which serve also as rest and protection piers to the ends of the draw-span when open. These guard piers are built of masonry, iron, or timber. They are also required above and below both pivot and rest piers in some cases, and when built of timber or masonry faced with timber act as guiding dikes for the passage of vessels and steamboats; when detached masonry guard piers are used, a floating crib or strong box is built between the guard piers and the pivot pier, which rises or falls with the water, being connected with the pier by properly arranged sliding surfaces.

96. The top dimensions are fixed by the bridge companies so as to allow ample room for the superstructure, but in general for piers the dimensions vary according to the length of span, from 6 feet by 20 feet to 12 feet by 40 feet, and the bottom dimensions of the neat work are fixed generally by allowing one inch for each vertical foot, but sometimes by abrupt enlargements in addition. Pivot piers are generally round, and vary at top from 20 feet to 30 feet, so as to allow ample margin for the turntable arrangements.

ARTICLE X.

RETAINING-WALLS.

97. Ordinary earth will not stand for any length of time with a vertical face, but will generally assume a slope more or less steep, according to the nature of the material, the angle of this slope is determined by the adhesion between the particles composing it and the friction between these particles or grains. The adhesion between the grains is destroyed by the disintegrating effects of air and moisture, therefore we may say that friction alone determines the angle of the slope. The angle which this slope finally assumes, measured from the horizon, is called the "angle of repose;" the slope itself is called the "natural slope." When an earth embankment either reaches this slope from natural causes or is built with this slope, its stability is insured. The effects of running water, from rain or other causes, will wash it in ruts and gullies, but this can be provided against by sodding, paving, or good drainage.

98. It is often necessary, however, to maintain a vertical face, as behind abutments when the approaches of the bridge are built of earth, as well as in other similar cases. It then becomes necessary to build a wall of some kind, called a retaining-wall, or in case of supporting the pressure of water, a reservoir wall. The principles of stability of these walls are the same.

99. The resulting force acting on a retaining-wall, or the abutment of arches, is always inclined to the vertical, more or less, depending upon the relative intensity of the weight of the wall acting vertically through the centre of gravity of its mass, and the intensity of the pressure of the earth on the wall, together with its direction. This obliquity of the resultant force causes two tendencies: the one is to cause the wall to slide upon the foundation-bed or upon some bed of the wall itself, the other is to overturn the wall bodily around some

axial line. The first tendency can easily be provided against by so arranging the foundation-bed, or some courses of the masonry, that their direction may be perpendicular to the direction of the force or pressure. A horizontal foundation-bed will generally give security against this tendency, unless the wall rests upon slippery and inclined layers of earth.

100. The tendency to overturn can only be resisted by sufficient thickness and weight of wall to fulfil the two following conditions:

1st. That the direction of the resultant pressure must not pierce the foundation-bed further from its geometrical centre than a certain limit, which may be taken at three eighths of the thickness. This point is called the "centre of pressure." This mode of stating the condition is a substitute for a factor-of-safety, as the actual point of overturning would only be reached when the direction of the resultant pressure passed through the outer edge of the wall.

2d. That the moment of weight of the wall with respect to an axis passing through the centre of pressure shall be at least equal to or greater than the moment of the outside pressure on the wall in respect to the same. This axis is taken as passing through the centre of pressure rather than through or along the outer edge of the masonry, for reasons of safety, as above stated, as the effect is to reduce the actual moment of the weight and to increase the moment of pressure.

101. This subject has been theorized and experimented on perhaps as much as any other engineering problem except that of arches. Formulæ are conflicting, owing to the uncertainty and variety of conditions actually existing. Many are the results of miniature experiments. Mr. Rankine evolves a formula purely from theoretical or supposed conditions,—all, no doubt, approximating the truth.

102. The requisite thickness of the wall is a certain fraction of the height. The practical result, however, obtained is that at any point of the wall, from the top to the bottom, the thickness of the wall must be not less than one third of the vertical height from the surface of the ground to that point, and need not in general be more than one half the height. Two fifths of the height may generally be taken as a safe thickness, all depending upon the nature of the material resting against the wall. If this material is in the nature of a fluid, such as water, quicksand, and the like, a greater thickness may be required—even equal to the height. If there is danger of the material being converted into a flowing mass by the presence of water, a good plan is to place a vertical (or nearly so) layer of broken stone or gravel between the material and the wall. This will serve to carry off the water, small holes being left through the wall to allow the water to escape. Retaining-walls sometimes bulge outwards without sliding on the foundation-bed or overturning. When such is the case the wall may be considered in a precarious condition, but new relations between the pressures arising therefrom may result in a condition of stability, and the wall may remain in its then condition for a long time.

103. The face of a retaining-wall is generally built of rough ashlar masonry, may be built of block-in-course or of brick, for two reasons: 1st. The main pressure is concentrated towards the face of the wall, and a better class of masonry is required. 2d. For the sake of appearances. The back of the wall is generally of a rough rubble, composed of large and small stones. The face and back should be thoroughly tied or bonded together, so that the entire wall may act together in resisting the pressure. The face of the wall is generally built on a batter, as in piers, but the back is almost always built in a series of steps of greater or less rise. This increases the stability of the wall, by bonding into the material behind, and having its weight increased by the weight of the natural material resting upon it. Some additional stability can be secured by inclining the wall backwards towards the pressure, or the same stability by this method can be secured with less masonry.

104. The face of the wall is built, as in case of piers, resting on the usual footing-courses, both to distribute the pressure over a larger surface, and at the same time to throw the centre of pressure further inward from the face of the wall.

105. When a retaining-wall is in the nature of a railroad abutment, or the abutment pier of arches, supporting a narrow embankment with the ordinary slopes, which generally are at the rate of $1\frac{1}{2}$ feet horizontal to each foot of vertical height, it is necessary to build at each end wing walls constructed as the face wall of the abutment, that is, with ashlar face and rubble backing. These wings can be built in the prolongation of the face wall, but decreasing in height, generally by a series of steps, as the slope of the bank descends; the total length of this wing at bottom would then be equal to $1\frac{1}{2}$ times the total height, the object being, by following with the masonry the slope of the embankment, to prevent the earth from falling in front of the abutment. Sometimes the wings run to the front and perpendicular to the main walls for a length determined by the circumstances of the case. This plan is rarely used, except in coming out of a tunnel, to support the sides of the open excavation; it of course adds immensely to the stability of a wall. Or the wings may extend, in case of an embankment, perpendicularly to the rear for a distance equal to from 1 to $1\frac{1}{2}$ times the horizontal base of the slop.e More commonly the wings make an obtuse angle with the face of the wall, depending upon the circumstances of the case. This plan is specially applicable to abutments on the banks of watercourses, where from the direction of the current there would be danger in times of floods of the water getting behind the abutments and scouring out the embankment. The angle between the main wall and the wing walls depending on the angle between the current of the stream and the direction of the embankment, and even when the directions of the stream and the embankment are at right angles, it has the advantage of presenting a funnel-shaped entrance and exit for the water, thereby relieving the danger of obstructing the free flow of the stream. The wings adding considerable stability to the main walls, these may not be as thick as required in isolated walls, resulting in a small saving of masonry on each abutment; and on a long line of road a little saved here and there amounts to an important item of cost. Main and wing walls should be finished with good large coping-stones, but these need not be cut or dressed as neatly as is the custom on piers.

106. In designing retaining-walls for the abutments of a bridge the steps on the back are so arranged that on the top of any step the thickness of the wall should be from $\frac{1}{3}$ to $\frac{1}{2}$ the height from the surface to that point; this is then carried up vertically for a certain distance, then another step is made, and so on. The back of the wall may be as rough as the builder pleases, provided the minimum thickness is maintained, and to avoid unnecessary care the builder always makes it thicker than required. On the front a bridge seat of from 3 to 5 feet in width must be provided for the end rest of the bridge, and back of this a wall, called a breast wall, must be built up to the under side of the crossties on the bridge; this is made from 2 to $2\frac{1}{2}$ feet thick and from 2 to 4 feet high, depending upon the length of the span and form of truss used, this information is obtained from the bridge company. The bottom dimensions are determined from this data, as in case of piers. In very high walls the centre of pressure on the foundation may vary materially from the centre of figure of the base, and care must be taken to keep it within the limits above prescribed in order to avoid too great unit of pressure on the base; this can be done by spreading the base with concrete and offset courses. (Figs. 7, 8, and 9.)

107. In order to prevent any tendency to slide, the condition of frictional stability must be fulfilled, which is that the direction of the resultant pressure must not make with the normal to any horizontal plane from bottom to top an angle greater than the angle of repose,—that is, than the angle at which the upper portion would slide on the lower. There is practically no danger of the sliding of one course of masonry on another, but the wall may slide as a whole upon its base; but in either event this tendency can be prevented by inclining the plane of the bed-joints so as to be nearly perpendicular to the direction of the resultant pressure, or the foundation bed can be cut into the form of steps. Good judgment can alone determine when these things are necessary. Cases have arisen when it was necessary to anchor the wall by the use of long

Fig. 8.

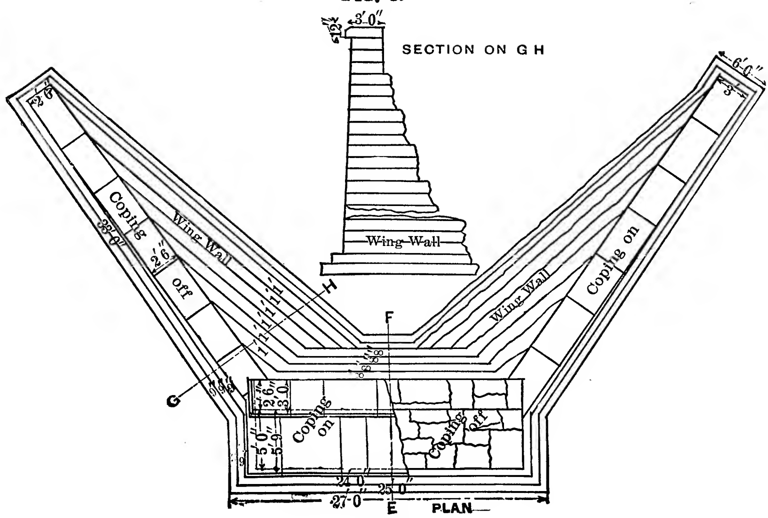


Fig. 7.

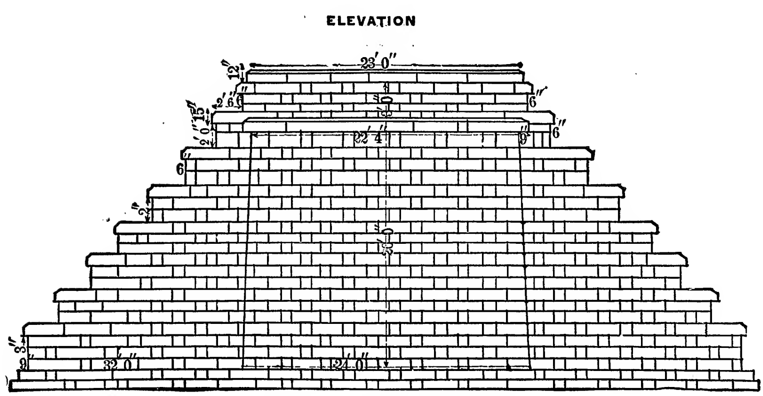


Fig. 9.—Wing Abutment.

[*To face page* 56.]

rods passing through the wall and fastened to iron plates or timber walls embedded in the ground some distance behind the wall, or by inclined struts in front resting at one end against the wall, and the other against walls embedded in the ground in front. These rods or struts should generally rest against the wall as near the bottom as convenient,—theoretically at a point $\frac{1}{3}$ of the height from the bottom.

108. The piers of a bridge on the Warrior River in Alabama built on and too near the sloping banks, without driving piles for them to rest on, had to be held in position by a system of strong struts as described above; and pneumatic tubes were sunk in an inclined position in front of the abutment piers of a bridge across the Schuylkill River in Philadelphia to prevent a continuation of a sliding discovered after completion of the bridge; and other instances could be cited, but withal it is rarely required.

In very high abutments large archways are often left under the wing walls, extending backward, for roadways as well as for economy.

ARTICLE XI.

FORMULÆ FOR THICKNESS.

109. The various theories and resulting formulæ seem either to be based on uncertain or erroneous data, and without undertaking to discuss or criticise these the writer will content himself with giving the geometrical representations of the conditions of stability and the common formulæ based upon the supposed conditions. Mr. Rankine assumes that the direction of the pressure is parallel to the surface of the ground, that its intensity is uniformly varying, and that its amount is represented by a prism whose base is a triangle, *sab*, Fig. 4, and whose length is the length of the wall. The triangular base is constructed as follows: From *a*, the bottom of the wall, draw $ab = x\frac{p_1}{p}$ parallel to the surface of the ground and also draw the line *sb*; then area of triangle $sab = \frac{1}{2}ab \times sc =$ also to vol-

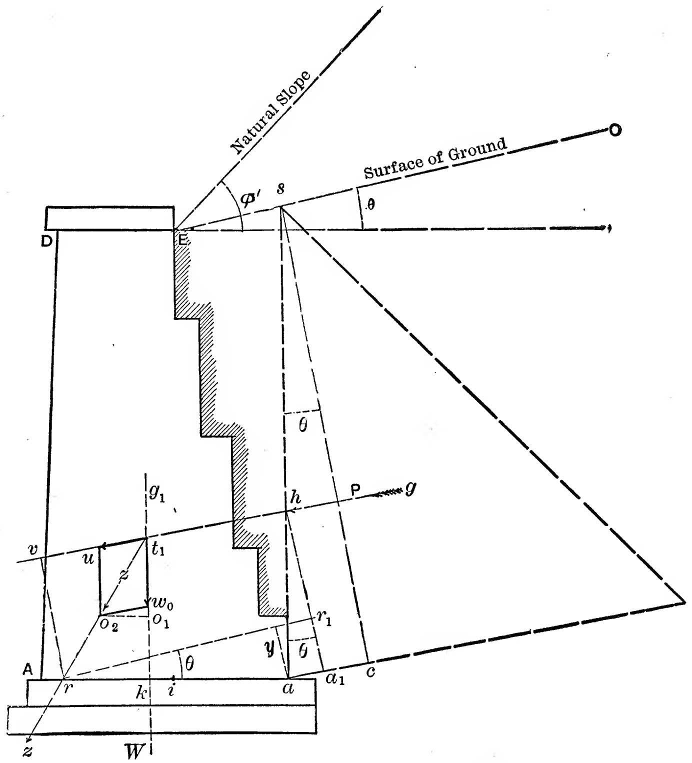


FIG. 10.—CROSS-SECTION OF RETAINING-WALL. PRISM OF EARTH PRESSURE. RESULTANT OF EARTH PRESSURE.

$EO =$ surface of ground;
$\theta =$ angle of slope of ground;
$\phi' =$ " " repose;
$sab =$ area of base of earth prism;
$as = x =$ vertical side of sab;
$sc = x \cos\theta =$ altitude of "

$$ab = x\frac{p_1}{p} = x\frac{\cos\theta - \sqrt{\cos^2\theta - \cos^2\phi}}{\cos\theta + \sqrt{\cos^2\theta - \cos^2\phi}};$$

$P =$ pressure of the earth $= t_1u$;
$W =$ weight of wall $= t_1w_0$;
$t =$ thickness of wall $= Aa$;
$g =$ centre of gravity of triangle sab;
$g_1 =$ " " " " wall;
$z =$ resultant pressure;
$r =$ centre of "
$g_1W =$ line of action of weight of wall;
$ri = qt =$ generally $\frac{3}{8}t$;
$ki = q_1t$ may be $+$ or $-$;
$ha = \frac{1}{3}x$; $ha_1 = \frac{1}{3}x\cos\theta$; $r_1a_1 = ay = (qt + \frac{1}{2}t)\sin\theta$.

ume of earth prism unity in length; and if $w' =$ weight of unit of volume of earth, weight of prism $= P = \frac{w' \times ab \times sc}{2}$. Moment of pressure of earth $= P \times vr = \frac{w'(ab \times sc)}{2} \times (ha_1 - r_1a_1)$. Substituting values given above, we have, for the moment of the pressure tending to overturn the wall,

$$P \times vr = \frac{w'x^2 \cos\theta}{2} \times \frac{p_1}{p} \times (\tfrac{1}{3}x \cos\theta - (q + \tfrac{1}{2})t \sin\theta). \quad (1)$$

The moment of resistance to overturning will be the weight of the wall + the weight of earth resting on the steps ($= W$) multiplied by its lever arm ($= rk$) $= W . rk = W(ri \pm ki) = W(qt \pm q_1t)$. The quantity q_1t will be positive or negative as the line of action of the weight g_1W is on the opposite or the same side of the centre of figure of the base Aa as the centre of pressure; in the Fig. 10 above it is negative. The moment of stability is thus $M = W(qt \pm q_1t)$, and this must be equal to or greater than the moment of the external pressure $P \times vr$; q being generally assumed $= \frac{3}{8}$.

$$W(\tfrac{3}{8} \pm q_1)t \geq \frac{w'x^2 \cos\theta}{2} \times \frac{p_1}{p} \times (\tfrac{1}{3}x \cos\theta - (q + \tfrac{1}{2})t \sin\theta). \quad (2)$$

The condition to resist sliding or stability of friction is that the angle $rt_1k \leq \phi$, the angle of repose of masonry on masonry, ϕ varying from 25° to 30°. Generally, all the quantities in eq. 2 are given except t, which can then be found. ϕ can be measured or calculated. In Fig. 10,

$$o_1o_2 = (o_1w_0 + t_1w_0) \times \text{tang } o_2t_1o;$$
$$o_1o_2 = P\cos\theta; \qquad o_1w_0 = P\sin\theta; \qquad t_1w_0 = W;$$

∴ we have

$$\text{tang } o_2t_1o_1 = \frac{P\cos\theta}{W + P\sin\theta} \leq \text{tang } \phi$$

as the condition of frictional stability. For fluid pressure

$$\theta = 0; \qquad \cos\theta = 1; \qquad \sin\theta = 0; \qquad \frac{p_1}{p} = 1.$$

Eq. 2 then becomes

$$w(\tfrac{3}{8} \pm q_1)t \geq \frac{w^1 x^3}{6}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

and

$$\frac{P}{W} = \text{tang } o_2 t_1 o_1 \leq \text{tang } \phi. \quad . \quad . \quad . \quad . \quad . \quad (4)$$

This whole theory from beginning to end is beautiful, and if the premises are true the conclusions are also. It is analogous to the pressure of water, and the formula is easily applicable to the pressure of water by making both θ and ϕ' equal to o, the formula then becomes

$$W(\tfrac{3}{8} + q)t = \frac{w' x^2}{2} \times \tfrac{1}{3}x = \frac{w' x^3}{6};$$

as

$$\frac{p_1}{p} = \frac{\cos\theta - \sqrt{\cos^2\theta - \cos^2\phi'}}{\cos\theta + \sqrt{\cos^2\theta - \cos^2\phi'}} = 1,$$

in this case. A practical example will be given in a subsequent paragraph.

110. The only other formula that will be mentioned may be called Moseley's. This is based upon the following conditions, namely, when a mass of earth is allowed to assume its own slope it will generally slide down until the slope makes an angle with the horizontal equal to the angle of repose, then it will have its natural slope; but if a wall with a vertical face be built to prevent this sliding, a pressure will be exerted against the wall by the tendency to slide. Experiment or theory or both show, however, that the weight of this sliding mass does not represent the maximum pressure, but if a plane be taken bisecting the angle between the natural slope and the back of the wall, the prism nearest the wall will exert the maximum pressure. The direction of the pressure is taken to be horizontal, and the point of application of the resultant pressure on the back of the wall will be at $\frac{1}{3}$ of the height of the wall from the bottom. Let AB (Fig. 11) be the back of the wall and vertical, BC the natural slope; then BD bisecting the angle ABC will be the plane of rupture, and the prism ABD will produce the maximum pressure. Hence, considering the length of the wall unity, the area ABD will also be the volume. Area $ABD = \frac{AB \times AD}{2} = \frac{x^2 \operatorname{tang} ABD}{2}$, and the weight W' of the prism equal to $\frac{w'x^2 \operatorname{tang} ABD}{2}$, and the pressure due to this is $P = \frac{w'x^2 \operatorname{tang}^2 ABD}{2} = W' \operatorname{tang} ABD$, and the condition of stability will be, as before,

$$W(\tfrac{3}{8} \pm q_1)t = \frac{w'x^2 \operatorname{tang}^2 ABD}{2} \times \tfrac{1}{3}x = \frac{w'x^3 \operatorname{tang}^2 ABD}{6} \quad . \quad (5)$$

in which W equal to the weight of wall and earth on the steps (assuming a plane of division along AB), to be determined from the cross-section of the wall; small w' = unit weight (1 cu. ft.) of the material supported by the wall; $t = FB$ = thickness of wall, and angle $ABD = \frac{90° - \text{angle of repose } (\phi)}{2}$. The lever arm of the external pressure $= \frac{1}{3}x = hB$; $AB = x$ = height of wall. The moment of stability $= W(qt \pm q_1 t)$, and the condition of frictional stability tang $vrt_1 = rt_1k = \frac{P}{W} \leq \operatorname{tang} \phi$.

III. Both of these formulæ are supposed to be based on erroneous or false assumptions, and consequently the results are not considered at all reliable. They are, however, approximately true if the material is clean sand. As a simple application of the formulæ and for the sake of comparison, we will assume the following quantities: $AB = x = 20$ ft. height of the wall, supporting clay in a fair or normal condition, surface AC

of earth horizontal. The wall of rectangular horizontal section and vertical section, BC the natural slope, BD the plane of rupture, ϕ the angle of repose $= CBH = 34°$; then $ABD = \frac{90 - 34}{2} = 28°$, and tang $28° = 0.53$. The line of action of the weight passing through the centre of gravity of the base, or rather centre of figure, hence $q_1 = 0$; weight of a cubic foot of

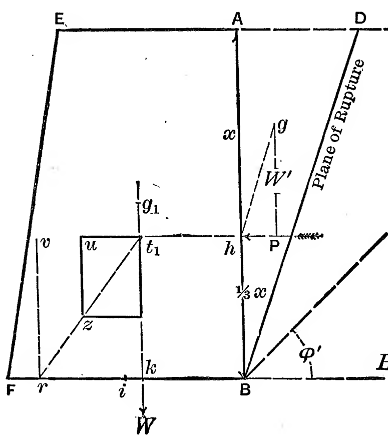


FIG. 11.—CROSS-SECTION OF RETAINING-WALL PRISM OF MAXIMUM PRESSURE RESULTANT.

clay $= w' = 120$ lbs., and $w =$ weight of a cu. ft. of masonry $= 150$ lbs. $W = wxt = 150 \times 20 \times t$. Substituting these values in eq. 5, par. 110, $W(\frac{3}{8} + q)t = \frac{1}{6}w'x^3$ tang^2 ABD, we have $150 \times 20 \times t \times \frac{3}{8}t = \frac{1}{6} \times 120 \times 8000 \times 0.2809$; hence $t = 6.3$ ft. = thickness of wall. Now substituting in eq. 2 (Rankine's),

$$W(\tfrac{3}{8} + q_1)t = \frac{w'x^2 \cos \theta}{2} \times \frac{p}{p} \times (\tfrac{1}{3}x \cos \theta - (q + \tfrac{1}{2})t \sin \theta,$$

recollecting that

$$\frac{p_1}{p} = \frac{\cos \theta - \sqrt{\cos^2 \theta - \cos^2 \phi'}}{\cos \theta + \sqrt{\cos^2 \theta - \cos^2 \phi'}} = \frac{1 - \sin \phi'}{1 + \sin \phi'},$$

since $\theta = 0$; $\cos\theta = 1$. And $\sin\theta = 0$, also the $\sin\phi' = \sin 34^\circ = 0.56$;

$$\frac{p_1}{p} = \frac{1 - .56}{1 + .56} = 0.28.$$

Substituting,

$$150 \times 20 \times t \times \tfrac{3}{8}t = \frac{120 \times 400 \times 0.28}{2} \times \tfrac{1}{3} \times 20, \quad \text{or } t = 6.3 \text{ ft.}$$

The formulæ reducing to the same value under these conditions, and as the angle of repose assumed is about that for dry sand, the formulæ give fairly good results. The least thickness in practice would be $\frac{1}{3}$ of $20 = 6.6$ ft., and more generally would be $\frac{2}{5}$ of $20 = 8.0$ ft. For wet clay or quicksand the formulæ give about 9.0 ft., but in practice it should not be less than 15 to 18 ft. These formulæ will generally give a less thickness than would be good practice. It will be observed that the axis about which moments have been taken is at a point $\frac{1}{8}$ of the thickness of the wall from the outer edge. If it had been taken at the outer edge the moment to resist overturning would have been a little greater, and consequently the resulting thickness would have been a little less than those obtained. In applying Rankine's formula for fluid pressure the only changes necessary would be in the value w' from 120 lbs. to $62\frac{1}{2}$ lbs. per cubic foot, and in making ϕ' and $\theta = 0$; then $\sin\phi' = \sin\theta = 0$, and $\frac{p_1}{p}$ would become unity. The substitution would give

$$150 \times 20 \times t \times \tfrac{3}{8}t = \frac{62.5 \times 400}{2} \times \tfrac{1}{3} \times 20.$$

$t = 8.6$ (should be 10 ft.) thickness of wall for water pressure. Water pressure can be calculated easily in any case by multiplying the area of the immersed surface by the depth to which its centre of gravity is immersed and by the weight of a cubic foot of water ($= 62\frac{1}{2}$ lbs). The point of application of resultant pressure is one third of the height of the wall from the bottom. The direction of the pressure is always perpendicular to the surface pressed, and this is true whether the surface is vertical, inclined, or horizontal.

112. In the case of surcharged retaining-walls, from the tops of which the slopes of the embankments rise at the angles of repose, such as terraces supported by walls near the bottom, or in masonry walls surmounted by embankments, as in the case of forts, even theory seems to be silent on this subject. The only practical rule is to be sure to make them thick enough. In placing the earth behind retaining-walls the material should be placed in thin layers and well rammed for at least 10 ft. back from the wall; it then may be dumped in the usual way.

113. If the material is likely to become like quicksand or soft mud, it is best to assume it as a fluid having the weight of the solid material and the angle of repose equal to zero, as in case of water; this will make the thickness from $1\frac{1}{2}$ to 2 times of that to support water.

114. Retaining-walls are of three kinds. Where the wings are inclined to the face of the wall, they are simply called wing abutments; where they extend back perpendicularly, with a hollow space between, they are called "U" abutments, the hollow to be filled with earth; and where a solid stem extends back, they are called "T" abutments. There is no advantage in the last, and requires more masonry, as a rule. The T abutments should be left hollow for about 2 ft. from the top, so as to allow sand, clay, or broken stone to be used for the cross-ties to rest on, as otherwise a jarring disagreeable motion will follow when a heavy rolling load passes. (Figs. 7, 8, and 9.)

ARTICLE XII.

ARCHES.

115. THE theory of arches is perhaps as little understood as in ages past. Some fall, some are doubtless in a precarious condition, some stand. Mathematical and mechanical theories, after carrying you through the intricate mazes of higher mathematics, have surely led to no satisfactory or practical results. The theory of graphics is more pleasant to handle, certainly easier to grasp, and may do admirably to back up guesses, or to shift responsibility in case of accident or failure. Mr. Rankine, after going through a most able and wonderfully conceived discussion of this subject, tells you to make your factor-of-safety from 20 to 40, and closes by saying : "The best course in practice is to assume a depth for the key-stone according to an empirical rule, founded on dimensions of good existing examples of bridges." We might inquire here how the old arch-builders came anywhere near safe dimensions Did they understand the theory of the arch ? or did they arrive by repeated failures or disasters to what at any rate are safe dimensions, and we now profit by their experience ? We must do the best we can, but be sure of being on the safe side.

116. Having fixed upon the depth of the key-stone, the same depth is maintained down to the springing line, in small arches. In arches of long span the depth increases gradually from crown to springing line, so as to maintain the unit pressure the same throughout, according to a simple and well-known law, that at any bed-joint the resultant pressure will be the hypothenuse of a right-angled triangle, of which the base is taken to represent the horizontal thrust at the crown and the altitude the weight on that portion of the arch ring from the crown to the bed-joint under consideration ; and by proper construction to scale on the drawing itself, the direction of the resultant and centre of pressure can be determined. At the springing the resultant pressure is represented in direction, magnitude, and point of application, which three elements must be known, by the hypothenuse of a right-angled triangle, the base being the horizontal thrust at the crown, the vertical being the weight of the half arch and any load upon it, supposed to pass through the centre of gravity of the mass. Following the process above mentioned for finding the centre of pressure at each bed-joint in the arch ring, the line passing through these centres of pressure is called the line of pressure, which for safety should be confined to the middle third of the thickness of the arch ring. Although after all this we may be in doubt whether the line of pressure will under all conditions remain where we put

it, yet we fortunately do know by experiment and observation the manner in which both flat and pointed arches give way; and though we may have blundered in determining the thickness of the arch ring and the proper curve to which it should be built, the above knowledge enables us to correct this error by what is known as the backing. Flat arches give way by breaking into four parts,—opening at the crown of the arch on the underside or the intrados, and opening on either side at a joint not definitely known, on the top or extrados, but never above that point which makes an angle of 45 with the horizon, the two upper parts falling inwards and pressing the two lower parts outwards. This last can be prevented by carrying up the masonry of the abutments above the point mentioned, and this in turn preventing the upper parts from falling.

117. In pointed arches the condition is just reversed, the two lower parts falling inwards and tending to lift the upper parts. This can be prevented by weight of sufficient magnitude on top; so, notwithstanding the ignorance on the subject of arches, by due precaution we can feel reasonably safe as regards the stability of any given arch.

118. The almost universal rule is to build the ring of the arch of the very best kind of ashlar masonry, cut so that the ring stones may bear against each other with the thinnest possible joints, which can be filled with grout or at least very thin mortar. The backing, the abutments, and the spandrels, or the wall resting on the arch ring, can be built of a less costly class of masonry.

119. Applying the principles explained in discussing retaining-walls, the direction of the pressure is towards the back of the wall rather than the face; hence the back of an abutment carrying an arch should be built of ashlar, or at least a good class of masonry. The exposed face need not be so good, but for appearance' sake it is generally ashlar; but in the case of arches the abutment is generally supported on the back by an embankment, and the same care is not necessary in building the back or unexposed face of the masonry.

120. In building an arch ring it should be built from both abutments at the same time upwards towards the crown, consequently the arch ring has to be supported until the arch is completed, that is, when the key-stone is put in. The framework, generally of timber, which supports the arch ring is called a "Centre;" this centre generally remains in place until the cement has had time to set, and is then removed. The frame generally rests on wedges; these being driven out gradually, the centre falls from the arch without shock or jar.

121. Arches are generally built over streams, roads, openings such as doors and windows in houses; and sometimes where large, heavy masses of masonry, such as unusually large piers or abutments, are to be built, requiring large quantities of masonry, the amount of masonry can be materially diminished by the use of arches, without injuring the stability of the structure. Where the stone of which it is built is strong and hard enough to bear the superincumbent weight on a considerably reduced area of bearing surface, and where also the foundation bed can bear the increased unit pressure, this can be reduced by the use of inverted arches under the arch proper. Stone arches of great span are not now built to the same extent as they were formerly, iron and steel having been substituted to a very great extent, and generally as horizontal trusses.

122. With no exact mathematical formulæ to guide us in the construction of arches, we are mainly compelled to follow some empirical rule, based upon the dimensions of existing arches, which at least stand though we do not know the amount or direction of action of the external forces or loads, even when fixed or dead, and still less of the effect of heavy rolling or moving loads. We can, however, approximate to these; and with the knowledge that the arch ring must give way either by crushing the voussoirs or arch stones, or by the sliding of one stone on another, or by the arch ring rotating or revolving inwards or outwards around the inner or outer edges of some of the stone as an axis, we can arrive at a safe thickness of the arch ring and the proper form of the arch; and in general we boldly assume the form of the arch ring and the thickness of it, and by a tentative process determine whether it will be stable under the conditions assumed.

We know the resistance to crushing of the stone,—this must not be exceeded by the greatest pressure to which it is subjected after allowing a large factor for safety,—and that it should be distributed as uniformly over the bearing surface of the stone as possible, and for this the centre of pressure should be as near the centre of the bearing surface as possible. To resist overturning around the edge of any jointthe centre of pressure must not be above or below the arch ring at any point, but must be on the surface of the stone, and as near the centre of figure as possible—at any rate within the middle third of the arch ring. To prevent sliding, the resultant pressure at any bed-joint must not make an angle greater than the angle of repose with the normal to the bed-joint at that point. The backing should be built up above that joint which makes an angle of 45 with the vertical: this backing is generally carried up to the crown, gradually thinning as it approaches the top. All of these conditions being fulfilled, the engineer may feel reasonably safe as to the stability of an arch; if, however, the graphical solution of the problem fails in any of the above respects, the arch ring must be made thicker or the form of the curve changed, or both.

123. It will be best to define the terms that will be used. The arch, taken as a whole, consists, 1st, of the abutments from which the arch springs; the top of the abutment on the inner edge is called the springing line; the truncated wedge-shaped stones resting on the abutment are called Skew-backs; 2d, of the arch ring itself, composed of wedge-shaped stones, called voussoirs or ring-stones, of varying sizes, but for the same arch the breadth should be as uniform as practicable; lengths should vary in order to get bond; the under side of the stones are cut to the curve of the arch. The under or cylindrical surface of the arch is called the Intrados or Soffit. The upper surface or back of the stones (generally left rough, conforming roughly to the curve of the ring of the arch) is the Extrados or back. The thickness of the ring is determined by the depth of the surfaces in actual contact included between two parallel curves, the intrados and extrados proper. The exposed under surface is generally dressed smooth; the joints between the voussoirs are always cut true. The bed-joints or surfaces of contact of the stones radiate from the centre or centres of the curves of the arch ring. The Key-stone is at the top of the arch or crown; it is the last stone put in, and the arch is not self-supporting until it is in place. The face of the arch is its end or head, the axis is the centre line, perpendicular to the head of the arch in square arches, and oblique in Skew arches. A ring-course is a portion of the arch ring included between two vertical planes perpendicular to the axis or parrallel to the head of the arch, and at any distance from each other,—say a foot or two. A String-course is that portion of the arch ring included between two inclined planes extended from end to end of the arch and intersecting in the axis of the arch, these planes containing the contiguous joints between the stones. The centres of pressure are the points in which the resultant pressures pierce the joints between the stones, and the line of pressure is the curved line passing through them. In large arches, especially when flat, the thickness of the ring-stone increases from the crown to the springing as the secant of the angle of inclination of the curve at any point, and at the springing may be $1\frac{1}{3}$ times that at the crown; in circular arches, if small, no increase is made generally.

124. A Full centre arch is a semicircumference in cross-section, and is rarely used except for comparatively small arches, as the rise would be too great, if for no other reason. The Segmental arch is flat, and generally a segment of one circle, with a long radius, and is called a one-centre arch; sometimes it is composed of segments of three circles, the upper part of a long radius, and the portions near the springing, called the Haunches, having short and equal radii: this approaches the elliptical form, and is generally called the elliptical arch; the true ellipse may be used. The span of the arch is the horizontal distance from the springing line to springing line. The Rise of the arch is the vertical distance from the springing line to the soffit at the crown.

125. The Spandrel Wall or parapet wall is not an essential part of the arch, but is built to give a finish to the ends, and at the same time if an embankment is built over the arch it serves as a retaining-wall for the foot of the slope of the embankment. It is a wall 3 or 4 feet high, built over the ends of the arch ring and in the plane of the arch, and is finished with a coping; it, with the wings, prevents the earth from rolling over or around the ends of the arch; it supports a surcharge embankment and should be thicker than that of a retaining-wall of that height. Sometimes intermediate walls are built parallel to the head walls. The backing between the head walls is really a part of the arch proper.

126. To determine the length of an arch, from end to end supporting an embankment above, the slope of the embankment being $1\frac{1}{2}$ to 1, it is merely necessary to deduct from the width of the embankment at the bottom three times the height from the ground line to the top of the spandrel wall; the arch should be a little longer than this difference. The wing walls then extend to the foot of the slope.

127. The masonry of the abutment is generally faced with ashlar, block-in-course, or coursed rubble masonry; theoretically, the back should be equally good or better than the face, but is generally of a rougher finish. Owing to the fact that the thrust of the arch tends to overturn the wall in one direction and the embankment in the other, the direction of the resultant may be inclined either way, or may be vertical according to their relative magnitudes. The embankment should be built on both sides of the arch at the same time, and should be rammed in layers around and over the arch for at least 10 feet in thickness, after which the earth may be dumped in the usual way. It is customary to pile up behind the abutments the shivers of rock and débris accumulating around the work. This facilitates the drainage, and at the same time strengthens the wall until the embankment is built.

128. In small arches, unless the bed of the stream is rocky, it is best to pave the bottom between the walls with stone from 6 to 12 inches thick, and also to build apron walls under

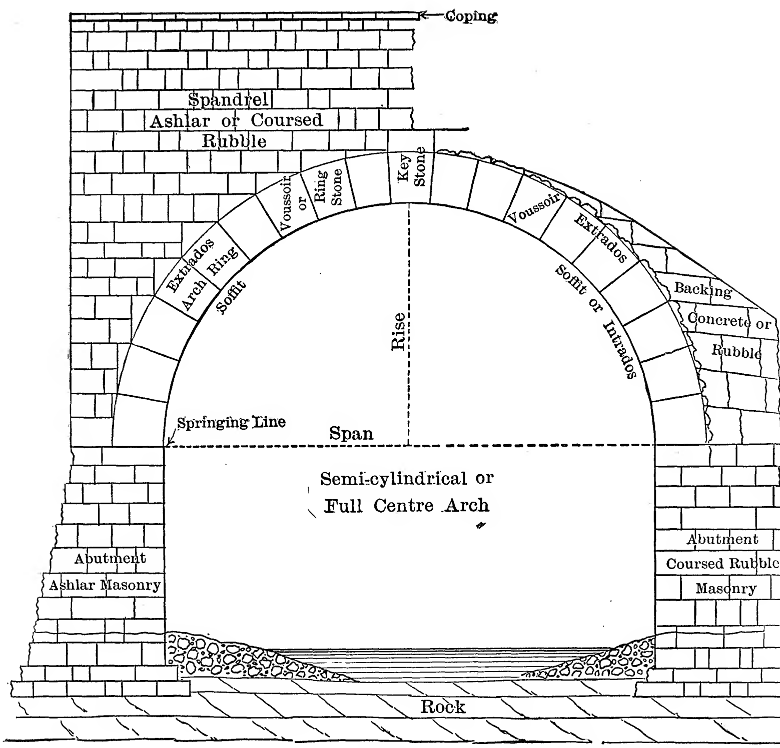


FIG 12. [*To face page* 70.]

the ends deeper than the foundation bed of the abutments, to avoid any danger of scouring. The spans for such arches generally vary from 5 to 20 feet, and are generally full-centre arches. For longer spans it will generally be economical to use the flat or segment arch, and avoid too great a rise of the arch.

129. The arch ring-stones are all dressed true on the joints and soffit, and are of best kind of ashlar. The width of the ring-stones is seldom less than one foot or more than three feet, and the thickness or depth from one to five feet. The upper surface is, in general, covered with a layer of cement or asphalt, or some waterproof substance to drain the water to the proper drains, and prevent the dripping that would otherwise pass through the joints of the ring. The stones of the arch ring-break joints in the direction of the length of the arch. Arch stones are often cut and marked so as to fit in a certain position, as shown on the development of the arch on paper; this is convenient, and saves time and trouble. The development of a square arch would be a rectangle, one side of which is the length of the arch, and the other side is the length of the arch ring itself, upon which the string-courses can be laid down to scale in their true positions, and arranged so as to secure the proper bond, and the arch ring should be built to correspond. The ring-coursed stone on the ends of the arch are cut on top to vertical and horizontal surfaces in order to let the spandrel wall rest true on the ring, and also to bond with it, also for appearances.

130. The masonry of the spandrel may be ashlar or block-in-course masonry, backed with rubble. The wings may be the same kind of masonry, and proportioned as in retaining-walls. The backing is generally of heavy rubble or may be made of concrete, rounding off towards the crown of the arch, and covered over with a layer of cement.

ART. XIII.

SKEW ARCHES.

131. THE skew arch is built of the same parts and of the same kind of masonry in the corresponding parts, but, owing to the inclination of the axis of the arch to the plane of the face, the string-course joints are curved, and each joint is of a different kind of curve—that is, a series of irregular spirals drawn perpendicular to the lines of pressure in different sections of the length of the arch, taken at convenient intervals; and, although the general conditions of stability are the same as in a square arch of the same span, the above directions of the bed-joints require every stone in the arch to be of a different size and shape, with all surfaces curved. Here the knowledge of descriptive geometry and stereotomy are required to determine the shape of the stone, and to construct the templets to guide the stone-cutters. This is troublesome and laborious, and requires great accuracy and care, as each stone will fit in but one position in the arch ring; the cutting is expensive, the building is troublesome and slow, the whole structure is costly; hence engineers avoid as much as possible the use of this arch, and have so modified its construction as to avoid these difficulties.

132. Only a general outline of the first method will be given, the method of constructing the development of the soffit will be found in Rankine's Civil Engineering, pages 450 and 451.

133. The first thing is to draw to a large scale the development of the soffit of the arch. A ring or wheel when revolved once develops a straight line, equal in length to the circumference of the wheel; a right cylinder, or take a semi-cylinder, when revolved develops a rectangle, the length of which is the length of the arch, and the breadth is the length of the semicircumference on the soffit of an arch; and an oblique cylinder, or the skew arch, when revolved will develop a figure ap-

proaching the rectangular in shape, with two straight parallel sides equal in length to the length of the arch, the other sides parallel, but curved, and equal to the length of the soffit.

134. A full discussion of arches, development of the soffit, lines of pressure, ring and string course joints, thrust or pressure at the crown and at other points, etc., will be found in another volume.

ARTICLE XIV.

DEPTH OF KEYSTONE.

135. THERE are many methods and theories on this subject, but as none of them lead to better or more certain or more reliable results, the reader is referred, for full discussions, to such authors as Rankine, Weisbach, Moseley. In practice empirical formulæ are used. Trautwine gives the following practical formula for determining depth of arch ring at the crown: Depth of key-stone in feet equal to $\left(\frac{\sqrt{\text{radius} + \frac{1}{2}\text{ span}}}{4}\right) + 0.2$ ft. for first-class cut-stone work. Increase this result by $\frac{1}{8}$ part for second-class work, and for brick or rubble $\frac{1}{4}$ part. The depth of the arch ring should increase, theoretically, from the crown to the springing, this increase at the springing being from one-fourth to one-half the depth at the crown, but is never necessary for small arches. Rankine's formula is: Depth of key-stone in feet equal to $\sqrt{0.12 \times \text{radius at crown}}$; and for an arch of a series the depth of key-stone in feet equal to $\sqrt{0.17 \times \text{radius at crown}}$. For tunnels, which generally have elliptical cross-sections, depth of key-stone in feet equal to $\sqrt{0.12r}$, in which $r = \frac{a^2}{b}$, in which a is the rise of the arch from two thirds to three fourths of the transverse diameter and b is the semi-conjugate diameter.

In soft and slippery materials the thickness should be doubled. Assuming a radius at the crown of 160 ft. and span 147.6 ft., Trautwine's formula gives depth of key-stone equal to 4.0 ft. and Rankine's 4.4 ft.; actual thickness 4.9 ft. This was a segmental arch. Again, by Rankine, an elliptical arch 30 ft. span, and rise $7\frac{1}{2}$ ft., calculated thickness 1.9 and actual thickness 2 feet; a 90-ft. span segmental arch, rise 30 ft., calculated thickness 2.88, actual thickness of key-stone 3.0 ft. About the largest arch built is the Cabin John aqueduct, Washington, D. C. Span 220 ft., rise 57.25 ft., radius at crown 134.25 ft., thickness of arch ring at the crown 4.16 ft., and at the springing 6.0 ft.; segmental in form. For depth of this arch at crown Trautwine's formula gives 4.1 ft., and Rankine's 4.0 ft. Therefore we may safely conclude that either formula gives safe results in practice.

136. To what extent and in what manner a heavy rolling load affects the line of pressure or the stability of an arch is not known, but in very large and heavy arches, or where there is a great depth of earth over the top, it probably causes no great change. Several feet of earth or ballast should be placed over an arch to prevent the effects of shocks from a rapidly moving train. Arches are built of masonry, iron, or wood; the same general principles are applicable.

The above conditions are necessary to prevent overturning around the edge of any joint. To prevent sliding at any joint, the direction of the resultant pressure must not make with normal to that joint a greater angle than the angle of repose or of friction of stone on stone; this is not likely to take place unless the abutment settles.

137. In the above considerations no account is taken of the tenacity of the mortar or its adherence to the stone, which would add materially to the strength of the arch.

138. The conditions of stability of the abutment is the same as that of a retaining-wall acted upon by a resultant pressure equal in magnitude, direction and point of application of the resultant pressure of the arch at the springing. By building the abutments in courses radiating from the centre of the arch, the line of pressure in the abutment would be a continuation of the line of pressure in the arch ring, this should be confined in the middle third of the abutment, and when the courses are horizontal is an approximate continuation of that line. The flatter the arch the greater will be the tendency to overturn the abutments.

ARTICLE XV.

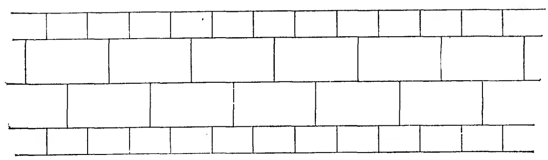
BRICK.

139. *Brick Walls and Piers* —Stone is always preferred for large piers and abutments, but in many parts of the country, especially in many Southern States, brick has to be relied upon for almost all purposes; and in all parts of the country brick is very largely used for private dwellings as well as for many public buildings. Brick can be called an artificial stone. The principal ingredients in brick are clay, sand, protoxide of iron. Other substances that may enter into ordinary clay either do no good or are absolutely harmful, carbonate of lime in any large proportions rendering the clay absolutely unfit for making brick. Sand should not exist in any excessive quantity. Protoxide of iron causes the red color in brick after burning, and also increases the strength and hardness.

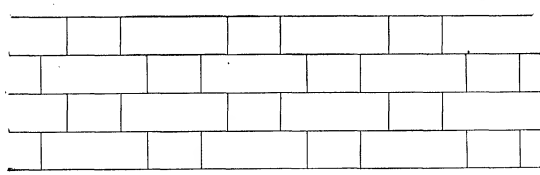
140. In making brick the clay is reduced to a state of rather stiff mud with water, then placed in what is called a "pug-mill," which consists essentially of a vertical cylinder, in the centre of which is a vertical shaft with radiating arms, so shaped and fixed that on turning the shaft, generally by a horse hitched to the end of a lever, the clay is thoroughly kneaded, and at the same time forced downwards to the bottom of the mill, where it is passed out of an aperture on to a platform, where it is then pressed into a mould of suitable size and shape. It is then placed on an open, well-prepared yard, where it is sun-dried for a short time. When properly dried (it constitutes something similar to the "adobe," which was formerly used for constructing houses) the bricks are built in a large mass or kiln of a certain established width and height, and of a length depending upon the number of bricks, varying from 100,000 to 300,000; eyes or flues are left at the bottom as receptacles for fuel—in ordinary cases wood. The bricks are laid rather open, so as to create a draft and allow the heat to pass in and around them. When ready the fire is started slowly at first and increased to an intense heat, and after burning for a period determined partly by the fuel used, but mainly by experience, the fires are allowed to die out gradually.

141. On opening a brick kiln after burning the quality of the brick may be divided into four classes, extreme outside brick, on sides and top being burnt so little that they may be thrown away as worthless; then a layer inside of the above, of more or less thickness, in which the brick are under burnt and soft; these are called pale or salmon brick, unfit for foundations or face work, but are used for filling in between good bricks in walls. In the centre of the mass forming the kiln a class of brick is found well burnt, hard, well shaped, and of good red color; this class of brick is good for any purpose. The lower part of the kiln just above the eyes are over-burnt, very hard, very brittle, and generally distorted, cracked, and even vitrified; these are not suitable for structures exposed to shocks. The second class are called pale, salmon, or under-burnt brick, very soft and porous. The third, body or red brick, hard and strong and used in face of wall.

142. Bricks are used for houses to a much larger extent than any other materials except wood. The walls of houses are generally carried up plumb or vertical, the dimensions at the bottom being determined by the nature and height of the walls, and purposes for which they are constructed,—warehouses, on account of the immense weight which may be placed on the floors, requiring thicker walls than dwelling-houses. This thickness in many cities is fixed by law, which doubtless corresponds with the practice very closely in all cities. The thickness is generally stated as follows: 8-inch or 9-inch walls being one brick thick, 12 to 13 inches being $1\frac{1}{2}$ brick thick, and so on. Bricks



English Bond.



Flemish Bond.

FIG. 13.

[*To face page* 77.]

vary a little in size in different parts of the country, but generally in the following limits: between 8 and 9 inches long, 4 to $4\frac{1}{2}$ inches wide, $2\frac{1}{4}$ to 3 inches thick. It will be noticed that the proportions of the several dimensions are about the same as for ashlar masonry. In ordinary houses, the thickness at the bottom varies, according to height, between $1\frac{1}{2}$ bricks thick (say 13 inches) to 4 bricks thick (32 inches), and decreasing to 1 brick thick at the top for houses of moderate height, and $1\frac{1}{2}$ brick thick for very high houses. This decrease is not made as in piers by a regular batter, but by abrupt changes or by offsets from story to story. The walls of warehouses should be thicker than the above, depending upon their size and the purpose for which they are used. The above dimensions refer to the body of the wall or the neat work; the footing-courses about double the above.

143. Brick-work is built in regular courses of the thickness of the brick, well bonded, and with joints of not over $\frac{1}{4}$ of an inch thick, or it is better controlled by saying that in a certain vertical height there shall not be more than so many courses of brick. If the brick is $2\frac{3}{4}$ inches thick, four courses should occupy a vertical foot on the face of the wall.

144. There are two kinds of bond, the English and the Flemish. It is probably immaterial which is used; but in the Flemish bond, where stretchers and headers alternate in each course, a more certain, uniform, and regular bond can be secured, a header being placed immediately over a stretcher below; whereas in the English bond the headers are in separate courses,—one, two, or more courses of stretchers, then one of headers, this proportion being regulated by the character of the work; a factory chimney, for instance, requiring a larger proportion of stretchers than headers, such as four courses of stretchers to one of headers. This kind of work should be rigidly executed, according to rules established both by theory and practice. More latitude can be given in the case of ordinary walls, but too much looseness and indifference is shown in this kind of work: so much so, that it is often impossible to say what proportion of stretchers to headers is allowed, it

being practically left to the masons to decide. Mr. Rankine says one course of headers to two stretchers gives equal strength longitudinally and transversely in the English bond.

145. Often in brick walls stone quoins or corner-stones are put in, these stones being, say, 10 or 12 inches thick, presumably to strengthen the corners, as well as for architectural effect; the policy in either case is at the best doubtful.

146. A great difference of practice in building walls of houses exists in regard to filling the joints, and the common practice is to make good beds; smear a dab of mortar on the end of the brick before placing it in position, leaving even the vertical joints of the bricks unfilled, and no attempt being made to fill the back joints at all; this practice certainly weakens the wall, even if it may have some compensating advantages.

147. The strength of ordinary bricks, such as the hard, red, well-burnt, is sufficient to resist crushing under any load that is likely to be placed on it in the walls of houses. This has been amply proved by experience in all parts of the country, and these bricks must have been made of clay varying largely in their composition, and both by hand and by many recently constructed machines. A few examples will suffice to establish the truth of the above. Mr. Rankine gives the actual existing pressure at the base of a chimney 450 feet high, 20,000 lbs. per square foot, or 140 lbs. per square inch. The brick shot-tower in Baltimore, 246 feet high, the pressure at the base is about 13,000 lbs. per square foot, or 90 lbs. per square inch, whereas 1100 lbs. per square inch is considered a fair ultimate strength for piers or walls of brick-work, giving a factor-of-safety of from 8 to 12. If good strong cement mortar is used, the ultimate strength can be taken at from 1500 to 2000 lbs. per square inch; the factor-of-safety will be at least from 11 to 14.

148. Brick piers and abutments are used to a large extent in the Southern States, on account of the difficulty and cost of securing stone of any kind. The writer built a bridge across the Tombigbee River, on the line of the Mobile and Birmingham Railway; the piers were of brick, resting on concrete, in the cribs of pneumatic caissons, a little below the water surface, the pressure at the bottom of the brick piers about 7600 lbs. per square foot of base; the brick were almost entirely obtained by pulling down old and abandoned warehouses in Mobile, Ala. The appearance of the brick indicated good strong brick, and the time that they had stood, without signs of wear or disintegration, indicated durability; in fact, they seemed to be superior to the new brick then being made.

149. Good brick seem to be as durable as any ordinary stone, can be built at a less cost per cubic yard than stone, and resists the effect of intense heat, such as resulting from fires in cities; is strong enough to carry any load likely to occur; is not affected by acid atmospheres; and now that brick can be obtained of different colors or variegated colors, it would seem that brick-work can satisfy all the conditions of strength, durability, and architectural effect desired.

150. One cause of the apparent distrust in brick-work is that manufacturers of brick are too anxious to sell a poor quality of brick, made of poor material, under burnt and soft, often very irregular in shape and size; angles not square, faces not parallel, and often badly warped and twisted. These defects, if confined to the backing or filling of the wall, would not be so objectionable; but masons are too apt to use these on the faces of the walls, causing ugly joints, irregular courses, a general bad and rough appearance; and in the filling they will use brick so soft that they are unfit for any purpose, and would soon return to the condition of mud if exposed. Often to get a sufficient quantity of brick you are compelled to take the general run of the kiln. These things cause a want of confidence.

151. But the high walls of houses in addition to resisting crushing, have to resist also the tendency to overturn, either from external forces from within or without. There is little or no danger of overturning from external forces, such as winds, as the floors, partitions, roofs, etc., prevent this; over-weighted floors might exert this tendency, but in this case the floors would have to give way to cause any dangerous effects, but at the same time there would be an outward tendency also. Both of these effects, whatever be their relative value, can be provided against by some simple device for anchoring the joists to the walls, which would at the same time give proper play for expansion, and give ventilation to the ends of the joist. One form of this is an iron casting with a rib at the bottom, a notch being made in the joist to fit over it. This anchoring prevents the walls from overturning outward in warehouses where large quantities of grain and other like material are stored. The roof also, in some cases, has a tendency to overturn the wall, as in the Gothic roof-truss, where no tie-beam is used. In all such cases the thickness of the walls must be increased, or it must be stiffened by buttresses. A standing wall of brick is considered the best and surest barrier to resist the spread of flames.

152. If walls of brick, such as house walls, give way by sliding, bulging, or overturning, the plane or line of breaking will follow the mortar-joints, as the resistance to the tendencies depends mainly upon the adhesion of the mortar to the brick, or the tensile strength of the mortar itself; therefore for the greatest strength the mortar used should be at least as strong as the brick itself. This can only be realized, however, by the use of cement mortar, as lime mortar will never be as strong as good brick, and as too often used is not much better than so much mud; often hardly enough lime is used to cover the grains of sand, and it is not unusual to see the mortar eaten out from 5 to 10 feet above the ground, apparently caused by water absorbed from the ground.

153. As to the adhesion of the mortar to the brick, it is hard to determine its value, as it depends to a very large extent upon the character of the brick and on the condition of its surface: if the brick is dry and dusty, the adhesion will be very small; if the brick is porous, clean, and wet, it will be of considerable value. It is inexcusable to lay brick unless they are

thoroughly wet or even saturated with water; but masons will not take this trouble unless compelled to do so, even in hot weather, and as a consequence the brick separates from the mortar with a perfectly clean surface. In the contrary cases it is often difficult, if not impossible, to remove the mortar from the faces of the brick. Always wet and keep the brick wet.

154. If what has been said is true, brick should never be used below ground unless good cement mortar is used, and it is always better to use stone even then. Dampness can be prevented from rising in the walls above the ground by using one or two layers of slate in the mortar-joints. Lime and cement can be mixed, using one barrel of lime and one barrel of cement, or even two of lime and one of cement would be vastly better than all lime.

155. There is a variety of hard strong bricks called compressed bricks: these are generally of good shape, square angles, true and parallel surfaces, made in different parts of the country; but these, owing to their great cost, are scarcely used except for facing walls, jams and lintels of doors and windows, cornices, etc. Walls faced in the ordinary way are hardly to be recommended, as in any case they are but poorly bonded into the back of the wall; this arises from several causes: the compressed bricks are not of the same sizes as the ordinary, and in addition it seems to be the practice to lay the facing with very thin mortar-joints, and to this add the ordinary carelessness in building the back walls. The monotonous red of these bricks and the unbroken uniformity in color does not always add to the appearance of such buildings.

156. A good safe rule for the thickness of the walls of houses would be not less than 12 inches at top, and an increase of 2 inches for each 12 feet to the bottom; this for a wall 50 feet high would be 20 inches at the ground line. It should seldom be thinner than this, and for warehouses and depots it should be from $1\frac{1}{2}$ to 2 times the above, according to circumstances.

157. Brick is also largely used in sewers, which are generally circular or oval in cross-section.

158. Brick pavements for streets have been used to a large extent in some localities and seem to give satisfaction, are claimed to be durable, easily cleaned, comparatively noiseless, and favoring a good foothold for horses. Bricks for this purpose should be hard and sound, and of the best quality, as they have to stand wear from both shocks and friction, to which ordinary structures are not exposed. For this purpose there is not perhaps sufficient experience or data to make a comparison with other paving materials.

ART. XVI.

BRICK ARCHES.

159. BRICKS are used very largely in building arches, especially over openings in ordinary houses, such as doors and windows, and at the bottom of walls to keep them from being pressed inwards, and at the same time they serve to distribute the pressure over the space between the walls ; in this case they are called inverted arches.

160. Ordinarily arch rings of brick consist of one, two, or more rings of brick, laid as stretchers, these rings being only held together by the adhesion of the mortar between them to the brick, and by the tenacity of the mortar. As seen above, the line of pressure in an arch is not always a symmetrical or regular curve, and consequently the entire pressure, or at any rate a large part of it, will be concentrated on one of the rings of brick, which might result in crushing the brick or in separating the rings. (Fig. 14).

161. There are only two methods by which this difficulty can be overcome : 1st, by having wedge-shaped bricks made especially for the purpose, which can either be equal to the thickness of the arch ring, or can be laid as header and stretcher, thereby distributing the pressure ; 2d, the arch can be so built, by regulating the thickness of the joints, that at intervals the radiating joints of the several rings shall be in the same plane, so that headers may be introduced, resulting in a distribution of the pressure.

162. Owing to the fact that arches, properly speaking, are built according to the curve of a circle or an ellipse or a combination of these, the outside rings will be a little longer than the inner rings, and as a consequence with ordinary bricks the joints will have to be a little thicker on the outside rings. This is regulated by laying the inner rings with a very little space between the bricks, and gradually increasing in thickness to the outer rings, or the increased space in the outer rings can be partly filled with pieces of ordinary slate, which answer well the purpose.

163. The strength of brick piers or arches can be materially increased by the use of ordinary hoop iron, bent into the joints and under and over the bricks: it is easily and simply applied, economical, and can be recommended; will also strengthen concrete and cement pipes. Wire netting is also used.

164. In the lining of tunnels which are arches, the side walls or abutments are continuations of the arch to the bottom, the foot of the walls being joined by inverted arches. Tunnels are generally lined with brick, on account of the ease with which brick-work can be built, especially in confined and cramped positions. Often tunnels are lined with timber; this, however, is only a temporary and economical expedient. This will be further alluded to under the subject of timber.

165. The thickness of tunnel arches can only be fixed by empirical rules, based upon the practice that has existed through the past ages, as the condition of the external forces are not thoroughly understood; but in case of tunnels, especially those at great depth, the pressure is practically uniform and constant, and the line of pressure is fixed and not altered by rolling loads, as is the case with arches built under ordinary conditions. The cross-section of a tunnel through ordinary earth requiring a lining is generally two-thirds to three-fourths of an ellipse; in rock it may be said to be of any shape most conveniently excavated, giving ample room for the purpose intended. Thickness of arching varies from 20 to 36 inches.

166. Arches are used largely for crossing streams, streets, roads, either over or under, of varying lengths and spans. The abutments of arches generally have wing walls constructed in one of the methods above described and for the same purposes. These wing walls are sometimes built on a curved batter; the principles of construction are, however, the same.

167. The principles of brick arches as to stability are the same as in stone-masonry arches. The line of pressure is constructed in the same manner, the depth of the arch ring can be found by the formula for masonry arches, but these results should be increased by at least 25 per cent; that is, if the formula calls for 2 feet of masonry, it should be at least 2.5 to 3 feet thick for the brick arch, but this is generally stated as so many rings; as the brick is placed flatwise as stretchers, each ring would be about 4¼ inches thick; this with the mortar-joints would take about 8 rings or courses.

168. Many large arches have been built of brick, but as a rule it is used mainly for very small arches, stone being preferred wherever it can be obtained conveniently and economically. It is not an unusual plan to make the end ring-courses of ashlar masonry, and between the two ends build the ring of brick. In order to secure a good bond, three or more string-courses of stone masonry could be used, the brick rings abutting against these stones; this is not, however, commonly resorted to.

169. Brick-work is estimated and paid for either by the cubic yard or surface measurement. In the first case it is usual to state that so many brick shall make a cubic yard; this is generally estimated at about five hundred bricks; it, however, depends upon the size of the brick and the thickness of the mortar-joints. In the second a square or perch on the face of a wall one brick thick is the basis of estimate; if the wall is two bricks thick, the surface is supposed double: this on the face of the wall includes openings either in part or entirely, according to the agreement.

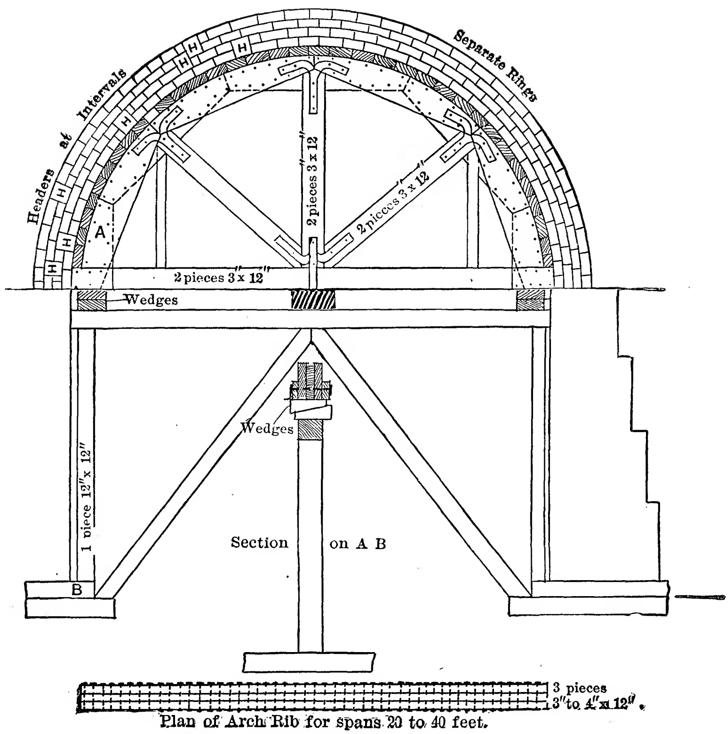


FIG. 14.—BRICK ARCH WITH WOODEN CENTRE IN PLACE.

[*To face page* 84.]

Art. XVII.

CONCLUSIONS.

170. We may therefore sum up as follows, in regard to the theories of the arch, and their practicable application:

171. The stability of the arch, as of all structures, depends upon the relations existing between the external forces or loads tending to produce strain, and the internal forces or stresses thus developed tending to resist or balance the external forces. In all discussions of walls or arches the length is considered as unity; that reduces the wall under consideration to the value of a section of the wall or arch included between two planes perpendicular to its axis at a unit distance apart, or simply equivalent to the area of the cross-section.

172. The external forces to be considered are the forces or loads acting upon the structure of whatever nature they may be, including the weight of the structure itself, and the supporting forces, whether applied to the whole structure, in which the supporting pressure is the resistance of the foundation, or whether applied to any portion of the structure, no matter how small into which it may be divided. In this case the supporting forces are the forces or stresses exerted between the portions of the structure under consideration and the other portions in contact with them: the conditions of equilibrium require that these shall balance each other.

173. A force is completely determined when its point of application, its direction, and its magnitude are fully known.

174. We are met, in deducing any theoretical formula for these relations, in the beginning, with a great want of knowledge as to either of these elements of force, and in fact the accurate determination is impossible.

175. As a consequence a great many suppositions have been made, and upon each supposition some theory has been constructed; and as the premises differ widely, so do the conclusions.

176. We do not know the pressure exerted by earth against a retaining-wall in either of its essential elements, and less do we know the pressure exerted upon an arch loaded with earth or other material, and in addition with heavy rolling loads. In arches, however, the assumption is made that the entire load above acts vertically, and with its full intensity upon the arch ring; this is certainly on the side of safety, eliminating all inclined or horizontal forces of any kind. The second assumption is that the arch ring supports this entire load: this naturally follows from the first. The third assumption is as to the point of application and direction of the thrust or stresses developed in the arch ring; these, however, being assumed, the magnitude of the thrust itself can be easily determined.

177. Every change, under the same external loads or forces, in the direction or point of application of the thrust gives an entirely different line of pressure, upon the position of which the stability of the arch is supposed to depend, and there may be any number of lines of pressure; the problem of determining the true line is evidently indeterminate.

178. The pressure at the crown is supposed to be horizontal, and must have its point of application in the arch ring itself, and generally in the middle third of its depth.

179. With these quantities assumed, together with observing the manner in which arches give way, we are enabled to determine with some degree of approximation the requisite depth and thickness of the arch ring for any given form and size of arch.

180. Arches give way either by crushing the voussoirs, or by the parts sliding on each other at some of the joints, or by the parts rotating either around the outer or inner edge of the arch ring.

181. To prevent crushing the arch stones, the intensity of the pressure must not be greater than the strength of the stone, and for safety not more than one-tenth of their strength. It should be uniformly distributed over the depth of the arch ring, or at any rate it should not vary from uniformity further than that which can be represented by the ordinate of a right-angled tri-

angle whose base is the depth of the arch ring, and whose height is double the mean pressure, found by dividing the total pressure on any joint by the depth of the joint (which is the area of a unit of length); in this case the pressure at either the intrados or extrados would be nothing. In other words, the greatest intensity of the pressure at any point must not exceed two times the mean pressure, supposing the total pressure to be uniformly distributed.

182. When sliding takes place, unless caused by settlement of one of the abutments, it will generally occur at four joints of the arch ring, splitting it into five parts: in flat arches the upper parts sliding downwards and two parts on either side sliding outwards; in pointed arches the upper part sliding upwards and the other two sliding inwards. To avoid this the resultant pressure at any joint should not make with the normal to the joint an angle greater than the angle of repose. Radiating joints will generally fulfil this condition; if not, the direction of the joints can be easily changed.

183. When arches give way by rotating or overturning around any joint it will generally occur at five points,—one at the crown, two at some point between the crown and the springing and two at the springing,—dividing the arch ring into four parts; in flat arches the two upper parts falling inwards or downwards, and the two lower parts outwards, and in pointed arches the reverse. This will be prevented by confining the line of pressure within the arch ring, and for perfect safety within the middle third. Wherever the arch ring opens is a joint of rupture, but we generally speak of the joints of rupture as applying to the joints on either side of the crown, between the springing and the crown. The exact position of the joint of rupture cannot be determined, but is supposed to be between those joints that make an angle from 30 to 45 degrees with the horizontal.

184. The general modes of determining the line of pressure graphically have been explained.

185. While building the arch ring it must be supported by a frame called the centre; this is generally made of timber.

For small arches it consists of an arch rib composed of two or more layers of plank, cut into short pieces of from 5 to 6 feet, so that when cut to the form of the arch they will be about 12 inches deep at centre and 8 inches deep at ends, with radial ends; these are bolted together so as to break joints; iron straps are generally placed over the joints and bolted. The upper surface is cut accurately to the form of the curve; this rib is connected with a tie-beam, which is generally two pieces of plank bolted to the rib, from which springs one or more vertical and radiating struts to support the rib. These frames rest on vertical supports; which are generally capped with timber, and between the tie-beam of the rib and the cap of the supports queen or double wedges are driven so as to bring the arch accurately to its proper position. These frames are placed at short intervals, depending upon the size of the arch and the strength of the frames. Over these frames and perpendicular to them are placed scantlings or laggings, so as to form a close sheeting to support the arch stones. In very large arches the centres are composed of strong timber bowstring girders, supported and braced by as many direct supports as practicable. It is necessary that the ribs of these frames should be practically rigid or unyielding, as a small yield or spring anywhere might result injuriously. The stones do not commence to bear on the centre until the joint is reached at which the stones would begin to slide on each other, and increasing then in a rapid ratio to the crown, and only becomes self-supporting when the key-stones are put in position.

186. A good part of the arch, from the springing on each side, can be built without the aid of centres, and by a liberal use of hoop iron, especially in brick arches, no centres need be used at all. Centres are not removed until the mortar has had time to dry.

ARTICLE XVIII.

BOX CULVERTS.

187. THERE is another structure, very small, and seemingly so unimportant that it is scarcely ever noticed, but at the same time used largely: this is the box culvert, which can be built of stone, brick, or timber, and is used to carry small streams under embankments. It consists essentially of two walls, 1, 2, or 3 feet apart, generally $1\frac{1}{2}$ to 2 feet thick and covered over the top with large flat stones; the height of the walls vary between 2 and 5 feet high; if larger than the above size should be required to carry off the water, it is usually built double, that is, two side walls and a middle wall, mainly on account of the difficulty in securing such large capping-stones as would be required. The ends can be left rough or neatly finished, and have small wing walls: its length in this case will be a little shorter than the total width of the embankment at the bottom; if no wing-walls and no spandrels are used, the total length must be equal to or greater than that width. At the ends an apron wall is built; a trench is dug two or more feet deeper than the foundation of the side walls, and perpendicular to the axis of the culvert, and built up with masonry, the object of which is to prevent the undermining action of the stream, the bottom of the culvert is generally paved with small stones; the embankment is then built over the culvert. Arches for the same purpose commonly have the apron walls, and are paved in the same manner. Arches are used when an opening more than 5 feet wide is required to pass the water (Figs. 15 and 16).

188. In filling over arches and culverts, special care must be taken not to endanger the stability by shocks; the earth should be deposited on both sides at the same time, thrown by shovels, and should be rammed in place; this should be done for about 10 feet on both sides and on top, after which the earth can be dumped on in the usual manner. This precaution is quite an important one, and should not be neglected.

189. For the purpose of carrying small streams under embankments, terra-cotta pipes, owing to their comparative cheapness, are now largely used in sizes from 6 inches to 2 feet A special bed of sand or fine earth should first be prepared to receive the pipes, otherwise they are likely to be broken or distorted by the weight; the earth over and around them should be carefully placed and packed; the ends should generally rest in masonry head walls of some kind; the joints should be filled with cement.

GENERAL PRINCIPLES.

Certain general rules or principles should be followed in constructing masonry structures.

190. The courses of masonry should, in general, be laid perpendicular to the resultant pressure; in ordinary cases horizontal courses will satisfy this condition well enough.

191. The vertical joints should not be continuous, but should be broken from course to course by overlapping the stones by a distance equal to the depth of the course, which in ashlar should not be less than one foot. This is known as the bond. A sufficient number of headers should be used to tie the wall together transversely, and these should be placed as nearly over the centre of the stretchers below as possible.

192. All joints and spaces between the stones should be fully filled with mortar.

193. Stratified stones should be laid on their natural beds, —generally known as the quarry bed.

194. The surfaces of porous stones should be moistened before being placed in position; this is essential with sandstone and brick. For appearance' sake, the largest stones should be placed near the bottom, the thickness of the courses gradually decreasing towards the top. A good rule for the lengths of the headers is to make them equal to $\frac{1}{3}$ the thickness of the wall at the point where placed, provided that they will not exceed 6 feet in length. All the above except the last applies equally to brick-work.

195. Good brick should be regular in shape, opposite plane

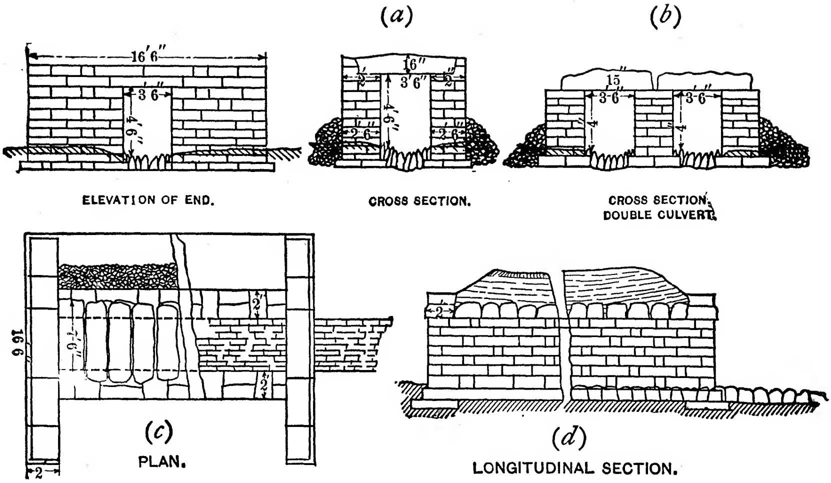


FIG. 15.—STONE CULVERTS.

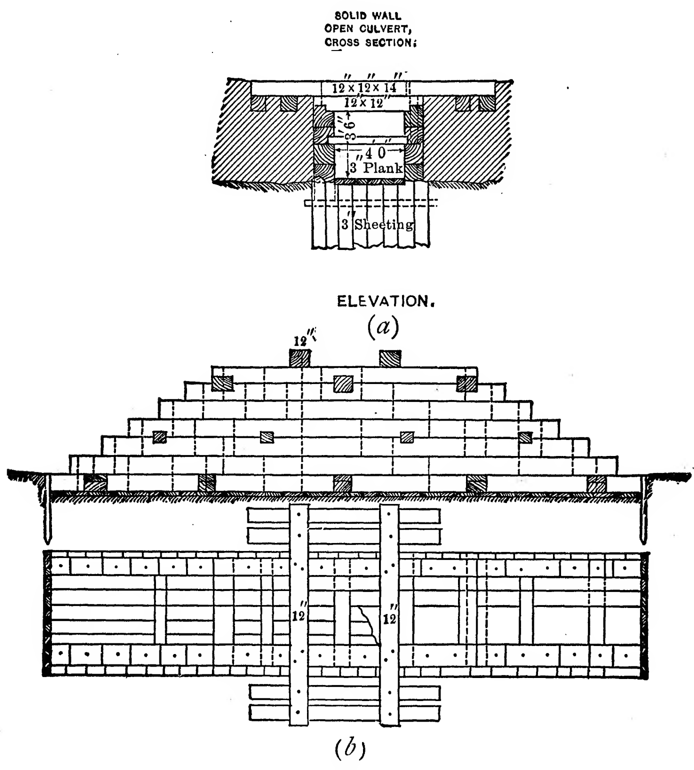


FIG. 16.—WOODEN CULVERTS.

[*To face page* 90.]

surfaces parallel to each other, and all angles right angles; should give a clear ringing sound when struck; should show on a broken surface a hard, compact and uniform structure; and should not absorb more than one fifteenth of their weight of water.

ARTICLE XIX.

CEMENT.

196. THE term Hydraulic Cement, or simply Cement, is applied to those substances, whether natural or artificial, which, when calcined and ground into powder and mixed with water, form a paste possessing the property of hardening under water. There is almost an infinite variety of these stones, found in layers or strata of different thicknesses, in the States of New York, Pennsylvania, Maryland, Virginia, Tennessee, and other States. These different strata, whether found in the same locality overlying each other, or in the same neighborhood, or in the different States, are found to be of different composition, and when treated in the same way yield products differing in a marked degree in regard to their hydraulic activity or rapidity of setting, and in hydraulic energy, or that property by which, whether they set rapidly or slowly, they attain a great and progressively increasing strength. Some take an initial set rapidly, but seem to increase in strength and hardness very slowly afterwards; others are slower in taking the initial set, but show a more regular and continuous increase in hardness than the first, and ultimately are far better. This difference is due not only to variations in composition, but also in a large degree to the degree of heat and the time consumed in the burning, so much so, that some of them only partly calcined possess little or no hydraulic energy; others, if at all overburnt, lose this property. In order, therefore, to produce a cement that will neither have too great nor too little hydraulic activity, and therefore better suited for ordinary purposes, the manufacturers mix the different grades of crude material, and obtain a product which is a more or less homo-

geneous mixture of the several grades. This may explain the want of uniformity so often found in the same brand of cement obtained at different times. Frequently, in works of great magnitude continuing through a period of years, requiring large quantities of cement, some cargoes show a marked difference in the time of setting; even with every precaution an ordinary batch of mortar will show signs of stiffening and setting before it can be used, resulting often, under strict inspection, in much waste, and again so slow in setting as to arouse suspicion that the entire cargo of cement is of an inferior grade. The writer has often seen some cements so quick in setting that they would stiffen to such a degree in the short interval required to lift the boxes and land them on the top of the pier, that it would be necessary to work the mortar again before using, the interval being not over 5 to 10 minutes, and in other cases after standing all night little or no appreciable change had taken place. The above cements are generally called the light quick-setting cements, weigh about 300 lbs. per barrel, and set in 5 minutes to 4 or 5 hours; are calcined at a moderate temperature, when 7 days old, 6 days in water, should have a tensile strength of not less than 60 lbs. per square inch and contain from 20 to 40 per cent of clay. These cements are generally known as the Rosendales, the Cumberland, Round Top, James River, Louisville, etc.; are found, respectively, in New York, Maryland, Virginia, and Kentucky.

197. What are known as the heavy, slow-setting cements are almost entirely artificial products, and are commonly known as Portland cements, such as the German, English, French, and American brands. They are composed of pure clay and lime containing from 20 to 25 per cent of clay, are calcined at a very high temperature, weigh about 400 lbs. per barrel, and should have a tensile strength of 180 lbs. when 7 days old, 6 days in water. These cements possess both great hydraulic activity and energy, and are far superior in every respect to the natural cements.

198. The temperature of the air and water have much to do with the setting of cements, and affect them in different degrees. As illustrating this, the writer noticed that Alsen's German Portland cement mortar was being delivered to the crib smoking and hot, and on being emptied from the box the mass fell to pieces as damp sand would do, and showing evidently an initial set. Upon inquiring into the cause, he found that for some reason hot water was being delivered through the pump; this happened on several occasions,—the time of passing from the mixer to the crib could not have been over 3 to 5 minutes,—and caused some considerable waste. This was ascribed by the men at first to what they called hot barrels, but was found to be due to the use of hot water. Concrete in the working chambers of caissons will set almost immediately, the temperature ranging from 80° to 90° Fahr., or more, and after standing 24 hours will require blasting to remove it. This was done in a caisson at the Schuylkill River. The result is the same whether mixed with hot water, immersed in hot water, or placed in a hot atmosphere. Some engineers require the cement and broken stone to be carried separately into the caisson and mixed below on this account. It may have some advantages, but the disadvantages would seem to be greater. The ingredients are not likely to be mixed as carefully or as thoroughly, they would be exposed for a longer time to the hot air of the working chamber, which is frequently very dry, and in addition will be more expensive. The writer always required one or two bucketsful of water to be poured into the supply shaft just before throwing in the concrete. This concrete was never mixed until a signal was given from below that they were ready for the concrete; the sand and cement were ready mixed, broken stone collected; it would then take only a few minutes to make the concrete, which was thrown immediately into the shaft. The compressed air passes rapidly into the shaft, the concrete drops on a platform and is immediately wheeled by barrows or shovelled into its place, deposited and rammed, the entire time consumed being not over 10 to 15 minutes. In this connection Gen. Gillmore mentions, page 81, that of two samples of cement paste, which set in 90° Fahr. in 1½ and 4 minutes, required at 65°, 6 and 17 min., and at 35°, 39 and 82 min., respectively, to get the same set; i.e., for a depression of temperature from 90° to 35° = 55°, the delay in setting to the same extent was $37\frac{1}{2}$ minutes in one case and 1 hour and 18 minutes in the other, and concludes that the presence of an excess of caustic lime in some of the varieties of cement causes them to be quick-setting, due to the heat developed in bringing this lime to a state of hydrate.

199. What is meant by a "set" is not well defined. Gen. Gillmore defines it as a state of the paste in which it will not change its form without fracture, or when it has entirely lost its plasticity;" this is evidently a vague and uncertain standard of comparison. Another test of the setting is the time that is requisite for the mortar to bear a small wire loaded with a certain weight,—a $\frac{1}{12}$-inch wire loaded with $\frac{1}{4}$ pound, and a $\frac{1}{24}$-inch wire loaded with 1 pound; and when the mortar will support these weights without indentation or depression it is said to have "set." The latter is purely a surface test, and as the temperature of the air and water play such an important part in determining the time of taking a set, its value is only for comparison of the hydraulic activity of two or more different brands, and would vary greatly according to the time of the year, and can be of but little practical value, as the cement may be tested at one time and used at another. If the tensile strength is taken as the standard, the Portland cements should be called the quick-setting, and the ordinary cements slow-setting. In practice, however, the distinction is not of very great importance, as most cements, when mixed in small quantities, afford ample time for using before any harmful change takes place in the mortar; but this is not, however, universal. A quick-setting cement has some advantages when exposed to immediate causes of deterioration or destruction, as when used in sea-water, works under such circumstances being constructed at low tide, and shortly flooded by high tide; under such circumstances a slow-setting cement could be used and faced, or protected by an inferior but very quick-setting cement.

200. Quick-lime is ordinarily slaked by pouring the entire quantity of water necessary on the lime at one time,—about two or three times the volume of the quick-lime. After slaking has commenced an addition of cold water is injurious. Good lime should not require stirring or the breaking of lumps during slaking. The same method is adopted in adding water to cement, but as this is mixed immediately before using, it is important not to use too much water, as the mortar will be too soft, and to avoid this the general practice is to mix the mortar at first rather dry, and then temper it with a small addition of water, to the proper consistency. This is preferable to making it too soft and wet at first, and then adding dry cement powder to bring it to a proper plastic condition.

The quantity of water necessary in mixing cement varies materially with the kind of cement used, the condition of the weather as to heat, moisture, or dryness, the age of the cement, whether dry or moist sand is used, and whether the broken stone is moist or dry. To form a paste of cement mortar of ordinary consistency 1 bbl. of cement will require about $\frac{1}{3}$ of a barrel of water, but when sand is used more water will be required; but this excess should be added in small quantities, as at a certain plastic state even small quantities of water will make the mortar too soft. An ordinary cement barrel contains $3\frac{3}{4}$ cubic feet of space, and about 5 cubic feet of loose cement can be packed in the barrels. Mixed in the above proportions there will result about $\frac{3}{8}$ of a barrel of paste. In some cases the sand is mixed with the paste, but the general practice is to first mix the sand and cement dry. These should be turned over and over until it has a uniform appearance. When carelessly mixed, patches or layers of cement without sand and sand without cement are readily seen, and if water is added in this condition it will be impossible to secure a homogeneous mortar. 1 barrel of cement, 2 barrels of sand, will make from 8 to $8\frac{1}{2}$ cubic feet of mortar, which will ordinarily be sufficient for laying 1 cubic yard of brick-work or hammer-dressed rubble-work, and in making 1 cubic yard of concrete. And 1 barrel of cement, 3 barrels of sand, will make about 12 cubic feet of mortar, sufficient for $1\frac{1}{2}$ cubic yards of ordinary masonry and concrete. Rough rubble will require from 11 to 12 cubic feet of mortar per cubic yard, or $1\frac{1}{2}$ barrels of cement per cubic yard; if mixed, 1 cement, 2 sand, or 1 barrel of cement; if mixed, 1 cement, 3 sand. 1 barrel of cement should be sufficient for $1\frac{1}{2}$ cubic yards of good ashlar masonry. For quick-lime mortar only about $\frac{1}{2}$ of the above is necessary, $\frac{1}{2}$ barrel of lime being equivalent to 1 barrel of cement. The above quantities are fair approximations, and will serve as a good basis for estimating the number of barrels of cement used in any proposed construction of masonry, whether ashlar, brick, concrete, or rubble, and consequently the cost of the same per cubic yard.

201. Mr. Trautwine, in edition of 1888, gives the following:

	Tensile strength, 7 days old, 6 days in water, in lbs. per sq. in.	Crushing strength, 7 days old, 6 days in water, in lbs. per sq. in.	Tons per sq. ft.
Portland Cement (neat), . . .	170 to 370	1100 to 2500	71 to 154
" " (2 sand), .	22 " 126	1100 " 2500	71 " 154
American " (neat), . . .	40 " 70	250 " 450	16 " 29
" " (2 sand), .	22	" " "	" " "

The practical engineer will rarely be able to do more than determine the tensile strength, as described in another paragraph. The above results seem to be low, and any cement not showing a strength equal to the inferior limits of 170 and 40 lbs. as above should be rejected. For Portland cement it is not unusual to specify that at the age of 7 days, 6 days in water the tensile strength should be at least 300 lbs. per square inch, and sometimes as high as 500 lbs., and for the American brands, such as the Rosendale, Louisville, etc., should certainly never fall below 40 lbs., and should be required to stand the superior limit of 70 lbs. in the time specified.

ARTICLE XX.

MORTAR.

202. MORTAR is a mixture of lime or cement, sand, and water in certain more or less definite proportions. It is usual to prescribe the proportions of lime or cement to that of sand as 1 to 1, 1 to 2, 1 to 3, etc, the proportion of sand depending upon the kind of cement and nature and importance of the work. Sometimes these proportions mean by volume, sometimes by weight, the amount of water being regulated somewhat arbitrarily. For ordinary walls quick-lime is generally used; for more important works cement: sometimes the two are mixed. The volume of mortar varies from one eighth to one third the volume of stone, the larger limit in concrete and rubble. It is stated on good authority that one volume of lime paste can be mixed with one volume of cement paste without material loss of strength, and that a mixture of lime paste equal to one half or three fourths that of the cement paste produces no appreciable injury, and is suitable for concrete when under water, and even better on many accounts. When not immediately immersed, it has the advantage of making some quick-setting cement slower in setting, and is certainly economical. The practice, however, differs, and it can be safely said that there is a very decided prejudice against this mixture.

203. Quick-lime is the product resulting from burning limestone in a proper constructed kiln, the heat driving off the carbonic acid, leaving white lumps and powder, which is the lime of commerce; this when mixed with water undergoes "slaking," a chemical action being set up, the water combining with the lime, which in the process falls to powder and results in a stiff white paste. The volume swells and great heat is developed. The perfectness of this process is probably the best and surest test of the quality; the presence of lumps or cores that will not slake means either an inferior quality of lime or insufficient burning, and results in great waste.

204. The proportions of sand and lime vary from 3 to 6 volumes of sand to one volume of quick-lime; the mortar thus procured is used extensively in walls of houses, and even in more important structures, but should never be used under ground or under water, as it will not attain any great degree of hardness under these conditions, if it hardens at all; that this is often done, only proves that our structures are many times stronger than required. Lime mortar should never be used in concrete in large masses, as it may be doubted if the interior of the mass will ever harden, as lime mortar sets alone by absorbing carbonic acid from the air, returning to the condition of a carbonate of lime, the cemented material then becoming an artificial sandstone. Cement mortars set by chemical action, and probably throughout the entire mass at the same time.

205. The ordinary limestone contains other ingredients such as silica, alumina, magnesia, in such small proportions, say not to exceed 10 per cent, that they exert no beneficial or harmful influence worth considering, but when we find limestone which contains these elements in proportions between 10 to 60 per cent, we find stones that possess peculiar properties, and of immense value and importance. The writer, in what he has said on this subject and what follows, does not propose to enter into the refinements of this subject, but to present a few facts which occur to him, of the greatest practical value. Those who desire can find a great deal of most interesting and valuable information, and doubtless the best available (up to time of publication, 1872) in Gillmore's book on limes, hydraulic cements, and mortars.

206. Cement stones exist in all states, from the very slightly hydraulic to the intensely so, culminating in the Portland cements, which are artificial products resulting from the mixture of pure clay and pure limestone in certain definite proportions determined by experiment, thoroughly mixed and calcined at a very high temperature, then ground to a fine powder. Portland cement is of course the best in every respect, but it is very expensive, and consequently only used in certain special structures of great magnitude and importance. For ordinary purposes requiring the use of hydraulic cement, the natural stone possessing hydraulic properties is calcined in a kiln, thoroughly ground and barrelled for use. These natural stones are found in many parts of the Middle States, each varying in some respect ; some very-slow setting, some very quick-setting, and other medium ; generally when ground of a mouse color, but some decidedly yellow. It cannot be denied that some very inferior grades are put upon the market, and without careful testing, are used, but as a rule the standard companies cannot afford to take the risk of such conduct, and can be relied on in the main.

207. As to different cement brands, we must first be guided by tests that have been made, and select that one which seems best suited for the purpose in view, and in addition simple test should be made on delivery. All broken barrels should be rejected, especially if it is to be stored for any length of time, as by exposure to the air it will take a set and be useless, and as it also results in waste. On opening barrels small portions found to be set will not necessarily indicate that the balance is ruined; but it excites suspicion of undue age or exposure, and means so much waste.

208. It is well to test a fair number of barrels by inserting the hand into the mass of cement, principally to determine the fineness to which it is ground, as a little experience will enable you to measure the sensitiveness of the touch; but this can easily be verified by obtaining a sieve with small meshes, numbered according to the number of meshes to the square inch. A No. 60 sieve would contain 3600 meshes to the square inch, a number 50 sieve 2500 to the square inch; this last ought to pass the entire quantity, except a small per cent; the coarser particles add nothing to the value of the cement, and amount to so much waste. In addition, small cakes of mortar made with good sand and cement in proportion used on the work will give a very good idea of its setting qualities either in air or water.

209. Where large quantities of cement are being delivered and used more or less rapidly, the above are the only tests practicable, and using the standard brands, it will determine practically whether the cargo under consideration is up to the standard. A medium, slow-setting cement is preferable, other things being equal, to a very quick-setting cement, as in large works the mortar will have to stand for some little time before being used entirely; this should be avoided as far as possible.

210. In general, you can rely on the Portland cements, German, English, or American brands, for any purpose; the Rosendale, Norton, and Hoffman brands, the Louisville, James River, Va., and many other brands have generally given entire satisfaction in the writer's experience. The writer has had but little experience with lime and cement mixed, but experiment as well as experience seem to authorize the use of it in the proportions above mentioned, and it certainly possesses the advantage of economy.

211. There have been comparatively few experiments made to determine the resistance to crushing of mortars when mixed with sand, and it has been the practice to determine the tensile strength of the many brands of cement, and from these results the crushing strength is, to a great extent, inferred, on the supposition that a high tensile strength indicates a high crushing strength. Many and varied experiments have been made in this manner.

212. Briquettes of neat cement, and mixed with varying proportions of sand, 1 to 1, 1 to 2, 1 to 3, have been tested. These briquettes have rounded ends, connected by a square prism of exactly 1 square inch cross-section; they are so formed that they will break at some point between the heads, and not at the junction of the head and neck. Brass moulds of the proper size and shape are made, and the mortar pressed into the mould and allowed to get an initial set. The mould is then removed, and the test is made on several briquettes made from the same batch of mortar, at intervals of 1, 2, 3, 7, 10 and more days. The testing machines are provided with nippers or clutches, and so adjusted as to take hold of the head exactly at the junction of the head and neck, and so situated as to make the pull exactly in a straight line, with no twisting or jerking, the power being slowly and gradually increased until the briquette is pulled apart; and it is generally specified on important works that the tensile strength should be so many pounds per square inch, in so many days after mixing. Machines and moulds are specially made for this purpose, and it would be useless to enter into any detailed description, as they can be more easily purchased than made to order; and in any event, unless the work is of very great importance, and cement is used in large quantities, the simple tests above alluded to will be satisfactory.

213. Pure, rich, or fat lime mortar hardens slowly in air by the absorption of carbonic acid gas, when used in comparatively thin walls, but it may never harden at all in the interior of large masses. It will not set or harden under water or in wet soils, and should therefore never be used under water under any circumstances, and it is false economy to use it under ground in damp earths. When water is added to lime it undergoes the processs of slaking; great heat is evolved; it swells to 2 or 3 times its original bulk, falls to a powder, and the resulting product is a hydrate of lime, unctuous or soapy to the touch, and forms a stiff paste. Lime should not be exposed to the air, as it will in time become air slaked, and materially injured. But when mixed in paste with sand, it is considered better to let it stand before using, and for this reason lime mortar is mixed in large quantities, left in piles and used as needed after stirring and tempering. Lime mortar shrinks considerably in setting.

214. Hydraulic limes containing from 10 to 20 per cent of silicates harden in air or water, but somewhat slowly under water; they slake to some extent, but slowly, and are supposed to harden by chemical action, probably through the whole mass at the same time, and shrink but little in setting. It should not be allowed to stand any great length of time after being mixed with water, as it will take an initial set, which, when disturbed by remixing, is supposed to materially diminish its ultimate strength.

215. Hydraulic cements, or simply cements, contain from 20 to 60 per cent of silicates. The proportion of silicates to the proportion of carbonate of lime determine the value of the cement. These vary considerably, and we have accordingly the heavy, slow-setting cements on the one hand. These are called Portland cements, and generally are manufactured, using pure clay and pure lime mixed in definite proportions determined by experiment. This mixture is then burned at a high temperature, ground exceedingly fine, carefully packed in good, strong, and tight barrels, generally lined with brown paper, in order to prevent any possible absorption of moisture; cost from 2 to 2½ times per barrel more than the ordinary cements; weigh considerably more per barrel, and, owing to their tendency to set quickly, should be mixed with water only a few minutes before using, and only in small quantities at a time. The more common brands are the German, English, and American, all of which brands are of excellent quality, and suitable for any purpose, and will stand two, three, or more volumes of sand, and seem not to be injured. These harden rapidly in air or water, and attain great ultimate strength.

216. The ordinary cements are obtained from natural stones, found in many parts of the country, and are known as the light, quick-setting cements; only weigh about two thirds as much, and take a very much greater time to set, and do not attain more than about half as much ultimate strength as the Portland cements, but are strong enough for almost any purpose, and, owing to their great abundance and relatively low cost, are used for all ordinary purposes, and can be more conveniently handled, as they take more time to set, but should not be mixed with water any great time before being used. The proportion of sand is rarely over 2 to 1 of cement. Some brands of these cements set much more rapidly than others,—so much so that some of them could be properly called slow-setting. All of these cements set well in air and in water.

217. If mortar, whether used alone or in concrete, is to be deposited under water, it should be allowed to take an initial set, as otherwise the cement will almost invariably be separated from the sand and the stone, no matter how carefully it may be deposited. Perhaps the best mode of depositing concrete under water is to fill open sacks or gunny sacks about two-thirds to three-fourths full of the concrete or mortar and deposit these in place, arranging them in courses, where practicable, header and stretcher system, and ramming each course as laid; the bagging is close enough not to allow the cement to be washed out, but at the same time open enough to allow the whole mass to be united and to become as compact as concrete itself. The writer used this method in the foundation of a pier over 100 feet high, and has also adopted this plan in other works of less magnitude, but never has the result been satisfactory when deposited under water in any other manner.

218. In whatever way mortar has been deposited under water, the result is at least uncertain; and there is positive evidence that in some cases where divers have examined concrete thus deposited it has been found to be far from homogeneous: deficiency of sand and excess of cement in some places and the reverse in others, and the same as to the stone and cement. Some experiments on a large scale were made by General Newton, using a very large box or caisson filled with water and depositing the concrete therein, with every precaution taken in order to secure favorable results; then subsequently pumping the water out, and removing the sides in order to make a thorough examination as to its condition. The mass was found to be far from uniform—an excess of stone in some parts, excess of mortar in others; and again excess of sand in places, and excess of cement in others. This could be doubtless avoided to some extent by allowing the concrete to attain some degree of set before being deposited in the water. That concrete has been deposited under water in many cases is undoubted, and structure erected on it which stand; but this does not fully justify the practice, and it should only be resorted to in cases of necessity, and generally the necessity can be removed by a little expenditure of money.

219. There is another manufactured or artificial mortar known as Pozzuolana, a substance of volcanic origin found in several countries, particularly in Italy, also other substances called trass or terras, having nearly the same composition, and composed mainly of silica and alumina, likewise of volcanic origin. When these substances are ground fine and mixed with the paste of rich lime they form a substance possessing great hydraulic properties, and equal in strength to the eminently hydraulic limes; sand is sometimes added. The proper proportions of these several ingredients would have to be determined by experiment. These mortars were largely used in marine construction by the Romans.

220. Artificial Pozzuolana is made by burning clay. Brick or tile dust when mixed with fat lime form a product possessing considerable hydraulic energy. "Forge scales, slags from iron foundries, ashes from lime-kilns, containing cinders, coal, and lime, are artificial pozzuolanas" (Gillmore). Some mixtures of these kinds seem to be good substitutes, when for any reason cement is difficult to secure.

221. These compounds as now made do not seem to stand the effects of sea-water, but some conflict and difference of opinion exist on this point. But our quick-setting cements made from the natural cement stones and the Portland cements can generally be relied upon to resist the solvent action of the sea-water, but they should be allowed to get a set before immersion.

222. It is a usual practice in cold climates to suspend masonry work of all kinds during the winter, as it is a prevalent opinion that the freezing of mortar unfits it for any ordinary purpose; in addition, work can never be done as economically in cold weather as at other times. As to the effects of freezing, opinions differ, it being maintained by some that although it may retard the setting, it has no ultimate injurious effects; others the contrary. Lime mortar, however, by best authority, is damaged when it alternately freezes and thaws, but not damaged when it remains frozen until it has set, and the same may be said of ordinary cement mortars. Portland cements are not affected even by alternately freezing and thawing.

223. The writer has been compelled on several works of importance to construct masonry nearly all the winter, and in several cases only stopped work when the masons refused to stand the exposure any longer, and in such cases he anticipated the probability of having to remove a part of the work in the spring. The stones had to be warmed and thawed out before using; and he has also used mortar mixed with hot water. And after the lapse of many weeks, through freezing and thawing weather, has found on examination that little or no damage was done apparently to the mortar on the top and exposed joints of piers thus abandoned; the mortar was powdered to the depth of a few inches, which being removed, the underlying mortar was as hard as could be desired in the time, and on no occasion does he recall that it was found necessary to remove any part of the structure. Mr. Trautwine makes substantially the same statement in his book. It is, however, undoubtedly best to suspend work in very cold weather.

224. The writer made a limited number of experiments, when building the bridge at Gray's Ferry, Philadelphia, in this direction using several brands of cement. Briquettes were made in the form commonly used in the test for tensile strength, of 1 cement, 2 sand, the proportion used on the work. One was kept in the house and occasionally moistened; this did not freeze at all. Another was frozen, then thawed out, and frozen again; this was repeated several times; and another was allowed to remain frozen for several days. These specimens were then tested to destruction in a suitable machine. The result was as follows: The one not frozen at all showed the greatest strength, the one that remained frozen came next, and the one alternately frozen and thawed gave the least strength. A sufficient number of experiments was not made to deduce any general law, or to eliminate those imperfections in the samples that might have existed, or to remove any irregularity that might have occurred in producing rupture when in the testing-machine, such as twisting or too rapid application of the weights, and only gives this for what it is worth.

225. Sometimes salt is mixed with the mortar to prevent freezing, but it is a question whether it will set at all. It produces a deliquescing, sloppy mass, inconvenient to handle, and remaining in this state for a long time; it is apt to disfigure the face of the masonry, but is frequently used. This method was used at the Susquehanna River bridge to a considerable extent. The masonry was only stopped on this work when the ice commenced to move, and boats could not be held in the river or stone delivered to the piers, and no evidence has existed of any damage to the mortar.

226. *Pointing-mortar for Masonry.*—Pointing masonry has for its object the protection of the mortar in the joints; to effect this the mortar should be cleaned out of the joints while soft to the depth of about 1½ inches, and this should be filled with a specially prepared mortar, and rammed as in calking; but in practice it simply means shaping the joints so as to present a neat appearance, and is often done to disguise an irregular-looking joint. The pointing-mortar should be neat cement, or at any rate not more than 1 sand to 1 cement, and before being applied should be allowed to take a set, and tempered with a little water when ready for use. Good pointing-mortar of 1 sand and 1 Louisville cement was used at Point Pleasant, the mortar being mixed the night before and allowed to remain over night, and tempered with a little water when used.

ARTICLE XXI.

SAND.

227. SAND is essential in lime mortar, as lime paste shrinks and cracks on drying, but is not in cement mortar, as cement paste does not shrink or crack on setting, but it is used for the sake of economy; it also increases resistance to crushing, but it diminishes the tenacity of the mortar, the proportions varying from 1 volume of cement to 3 of sand, to 1 of cement to 1 of sand, and in some cases more sand is used, but can hardly be said to be a good practice; the common practice is 1 of cement to 2 of sand by volume. Much has been said and written about sand, but ordinarily we have to do the best we can. Pit sand is generally angular, but apt to be dirty; river sand, the grains are apt to be rounded, and may or may not be dirty. As to the size of the grains, opinions are conflicting, for the best sand we may say that it should be clean; this is generally determined by rubbing it in the hand when damp: if it stains the hand it is loamy, and should be avoided. The

grains should be sharp; this is determined by rolling the sand in the hand; a well-defined grating sound indicates sharpness of grain.

228. Sand is used with grains varying from the size of a pea to a very fine grain, and purely as a practical question it would seem to be immaterial what size is used, provided the grains are not so large as to cause the stones to ride upon them, and to avoid this danger the sand is generally required to be screened. The size of the meshes of the screen depend upon the purpose for which the sand is used, but are commonly not over one eighth of an inch square; this will not pass a grain much over one sixteenth of an inch square, as the screening is ordinarily done in practice, which would not be objected to for any kind of work; this would, however, be called a coarse sand. Other sands vary even to almost imperceptible powder; but if it is really clean sand the grating sound can still be detected, and the distinct grains easily seen by an ordinary magnifying-glass; but this very fine sand is also objected to, and a medium grain seems to give more satisfaction. Damp sand if clean when pressed in the hand will not hold its shape on opening the hand, but will split and fall away; this is probably the best test as to the cleanness of sand, as the presence of clay or loam would cement the grains together. It is claimed that the finer the sand the more cement is required to make the mortar. Sand from salt water is objected to by many, unless it is well washed before using. Sand is generally composed of different-sized grains, which is a favorable condition for economy.

ARTICLE XXII.

STABILITY OF PIERS.

229. PIERS can give way by sliding along some horizontal bed-joint, or by overturning around some edge of the masonry, or by crushing the material of which it is made.

230. The pressures tending to cause sliding are the force of the wind, the force of the current acting directly on the end of the pier and crib, or acting on a mass of ice or driftwood, which sometimes accumulates above the pier for a greater or less distance on either side, and often extending from pier to pier. In such cases the ice or drift sinks and collects in very large masses, presenting a large exposed surface to the current; the effect of this is to cause the current to be much more rapid underneath the compact mass, endangering the destruction of the pier by undermining or scouring, and at the same time largely increasing the pressure on the piers.

231. The writer has observed closely and anxiously the action of a large mass of drift and ice under varying conditions, and a brief description of these cases, differing entirely, may not be uninteresting, and in some degree instructive, though he is unable to add anything in the way of a formula or law at all practical or useful. The action may be due to the ice and drift while stationary, or moving as a whole or in detached masses.

232. *First, while Stationary.—Expansion and Contraction of Ice.*—Ice on the surface of a lake may exert an enormous force, sufficient to move heavy masonry piers, caused by alternate expansion and contraction under changing temperature; even if the pier is protected by a sloping surface at the water's edge; the adhesion of the ice to the stone may be so great that it will exert against the pier a thrust due to its full crushing strength before it will fracture. The crushing strength of ice as given by different authorities varies very greatly, depending on its thickness, purity of water from which it is formed, and also with its temperature, the limits being from 400 to 1000 lbs. per square inch. This enormous pressure, acting over a considerable surface and often with a long lever arm, may exert a very great overturning moment. It is stated in *Engineering News*, Jan. 12, 1893, that a pier weighing 1000 tons was not only lifted, but held up under passing trains; and piers built on pile foundations were thrown out of line from 2 to 12 inches, the ice being from 10 to 12 inches thick. When the ice was cut away the piers moved back nearly to their original positions. This effect was attributed solely to expansion of the ice sheet. The writer's observations during a very severe winter at Havre de Grace, on the action of ice formed on the Susquehanna River against piers, does not correspond with the above estimate of the force of adhesion, and he does not understand how the ice can adhere to the pier at all unless the water is perfectly still; any oscillation whatever of the surface causing alternate rising and falling of the ice-sheet grinds and breaks the ice in contact with the surface of the pier, consequently preventing time for any adhesion to exist. This was noticeable on four or five piers at the Susquehanna Bridge; the ice was some 15 inches thick, and remaining for months, without movement in a horizontal direction, during great changes of temperature. It was unsafe to approach the piers too closely, that is, within a foot or two. The writer does not question the great pressure that would be exerted by the ice on the pier while expanding, except in so far as this condition immediately surrounding the piers would affect the pressure on the piers from expansion, as this effect can only appreciably exist when the ice sheet has a very great expanse. The sheet at the Susquehanna extended between two and three miles in the direction of the longer horizontal axis of the pier, but only, of course, a short distance, about 500 feet, in the direction of the shorter axis. Even, however, assuming a strong adhesion of the ice to the masonry of the pier, a wide range of temperature, a great expanse of ice in one or more directions, the danger arising from expanding ice can be economically avoided by cutting narrow channels through the ice parallel to the axis of the bridge.

233. *Secondly, while Moving as a Whole or in Detached Masses.—The Breaking and Flowing of the Ice at Gray's Ferry, Philadelphia, in the Schuylkill River and the Ohio River at several points, and the Drift Gorge on the Tombigbee River, Alabama.*—In the Susquehanna at Havre de Grace there is a broad stretch of very deep water divided into two channels by Watson's Island, extending about 2½ miles to a point above at Port Deposit, where rocky ledges seem to rise abruptly, a large part of which is exposed at low water; this continues for miles above, constituting the falls or rapids. Below these the current is comparatively slow, hardly perceptible in low water; this water freezes to the depth of two or more feet, and the ice rising and falling with the tide, is generally broken for several feet along the shores or around obstructions, such as piers, thereby free to move in a body. When the ice breaks above in the spring rises, it is brought down by the rapid current, and coming in contact with the solid mass of ice at the end of the rapids, it is thrown up and down and sidewise, flooding the streets of Port Deposit with water and masses of broken ice on one side, and the tow-path of the canal on the other side, of the river. Under this immense power the entire mass of ice, three or four miles long and more than half a mile wide, moves as a whole,—not more than 8 or 10 feet, probably, —crushing everything, except masonry, in its movement; this seems to relieve the pressure to a great extent, but the rapid flow of water under the ice can easily be observed. The ice will then remain in this position for some time.

234. It will have, however, crushed or broken or torn up any temporary breakwater made of large numbers of piles, and crushed into splinters the timber of the coffer-dams, breaking and twisting strong iron bolts and rods two inches in diameter; where it has struck the cutwaters of masonry piers, it will be split from 50 to 100 feet above the pier, the ice mounting the pier in great masses. Evidently the broken ice again begins to be accumulated at the upper end above and below the broken sheet of ice, and commonly believed to reach to the bed of the river; but this can hardly be, as in this case the water would flow for some distance over the top of the ice before such an immense sheet could adjust itself to the new conditions, which does not occur. At this time another movement takes place, with similar results as before; and this may continue for some time, alternately moving and stopping, the main sheet of ice still remaining solid, and in the writer's observation only breaking up when the warm weather has simply rotted it. He believes, therefore, that the moving force is the action of the current upon the large face at the upper end of packed ice both

above and below the main sheet, and that it is a mere question of the ice or the pier giving way. A square-ended pier would under such circumstances be put to a severe trial, but a good cutwater ploughs through the ice with hardly a tremor, aided as it is by the ice rising on the sloping cutwater and splitting in almost a straight line.

235. The moment of this force cannot be accurately or approximately estimated; the strongest kinds of timber frames, faced with iron rails, are simply crushed and splintered into kindling-wood.

236. Mr. Weisbach and others give formulæ for the amount of force exerted; but the elements of it are unknown, and all that can be done is to assume a certain value for the area of surface pressed, velocity of current, and some unknown factor depending on the shape of the pier, and obtaining a result which could have no practical value.

237. By observing the actual result and determining the resistance to splitting of the ice for a distance above the pier, we will have part of the resistance to the moving force. Much of it, however, is taken up by friction of the ice as it moves along the shore lines, wider at some and narrower at other parts. The writer has therefore simply determined the extent of the force on the pier by multiplying the area of the split surface by 10 tons to the square foot, this figure being simply assumed (it may be very far from the correct value, as so much depends upon the temperature and the condition of freezing; from some actual experiments the tensile strength has been found to vary from 140 to 200 pounds per square inch), in order to illustrate the manner of arriving at the actual force exerted.

238. The same may be said of accumulated driftwood, except that there is no means of arriving at the force necessary to break through a large mass of drift. This generally occurs in those rivers in which the water gets out of its banks and spreads over the country, which is the case in almost all southern rivers, many of which rise rapidly in one, two, or three days to the height of 40 to 50 feet, collecting, from miles on either side of the river, in places, immense amounts of drift, entire trees, logs, brushwood, etc., which form, on meeting an obstruction, a mattress dam from pier to pier, and to a depth of 10 to 12 feet, with logs and drift at all conceivable angles, extending from bank to bank and covering acres of water above.

239. In the Tombigbee River this was observed in a thirty-five-foot rise, but a timber coffer-dam (around a pivot pier built of brick), of verticals and two courses of 3-inch plank held down to the crib by 2-inch iron rods, octagonal in plane, well braced on the inside, resisted this pressure without springing a leak of any consequence. This could not have stood an ice movement such as above described, though apparently it looked equally as formidable, and would not yield. All that could be done was to keep a number of men with iron-pointed poles standing on the lower end of the drift gradually working piece after piece from the mass, and steam-tugs pulling with hawsers at the larger and longer pieces; and occasionally under this action it would be so far disentangled that immense masses would be carried by the current between the piers.

240. At the Schuylkill, after a very cold winter, the river was frozen solidly over, but owing to the number of bridges above, and the sinuosities of the stream, there was little chance for a similar movement of the ice to that at the Susquehanna, and in the break-up in the spring rise it was simply a rapid flow of immense masses of broken ice. This did not gorge to any extent; and though it broke barges loose strongly anchored and secured, upsetting some, carrying others off, and this flow continuing for several days acting on a square-end coffer-dam bolted to cribs, the coffer-dam being 20 or more feet wide and 12 to 15 feet high, with no masonry built inside, but good strong timber bracing being in place, the water rising nearly to the top of the dam, no damage was caused to the dam at all, did not fill with water, and work proceeded at once.

241. This dam had no protection of any kind above it, whereas the Susquehanna dam of the same construction, though double the size, was protected by a number of piles, numbering 75 or 100, driven close together in the form of a breakwater, which was crushed to pieces, and we never saw or heard of the piles afterwards. These piles were not braced rigidly together, as it was vainly thought better to allow the piles to gradually yield to the pressure, and thereby break the force of the pressure to some extent; but to no purpose.

242. In the one case the movement was slow and powerful; in the second no movement of the drift as a whole observable, notwithstanding the very great velocity of flow of water; and in the third large masses of broken ice moving with a very great velocity, their comparative destructive effects being as above described.

243. The action of the wind upon the piers has been explained and illustrated by an example. Experiment proves that its force, either alone or combined with the one above discussed, will neither cause ordinary piers to slide nor overturn. The piers of the P., W. & B. bridge at Havre de Grace are apparently unusually light, only extending a few feet above high-water, and carrying very short spans, giving what might be called the minimum elements of resistance to these pressures. These certainly seem safe against such pressures. It is true that they are protected by the island just above to a considerable extent.

244. As to the crushing resistance of the masonry under the influence of the weight of the pier, the wind, and the ice, there is such a wide margin of safety in the strength of the stone that no apprehension of this kind need be felt, as in practice the greatest pressure that can possibly occur would not exceed the mean pressure more than about two times, which would give a large factor-of-safety.

ARTICLE XXIII.

WATER-WAY IN CULVERTS.

245. IN determining the necessary water-way for box and arch culverts, so many and so varying conditions arise that it is impossible to deduce even an approximate formula that would be applicable to even a single stream, and on a hundred miles of road every ditch and every stream would require a

different coefficient, the value of which could only be wildly guessed at without a survey, the expense of which would be many times more than the cost of a culvert much larger than actually necessary. The more common formulæ are simply based on the supposition that the area of the water-way in square feet is equal to some root of the drainage area in acres multiplied by a constant the value of which is unknown; for instance, area of water-way in square feet

$$= C\sqrt{\text{drainage area in acres}}$$

(Myers' formula).

246. Practically no formula is generally used. The engineer can generally find in the neighborhood some highway bridge which by observation or information obtained from residents will serve as a guide; or in the absence of this, places can be found in which the water is confined between the banks of the stream in times of the highest flood known in the locality, or by taking cross-sections of the stream as indicated by the limits of high-water, but here the velocity of discharge would be unknown. Altogether we have no data upon which to base in the beginning an intelligent opinion, but in general, owing to the rapidity of railway construction now, temporary openings are left in the way of trestles, which can always be made sufficiently long to give full water-way, leaving the question of permanent culverts to be settled later.

247. Good judgment generally determines the proper size of culverts. Sometimes a single opening of a foot square built of rough stone will carry the necessary water, throwing, over and around this, broken stone (before dumping the ordinary earth), which would allow any unusual discharge to find its way through without any damage to the embankment, or small terra-cotta pipes from 6 inches to 1 foot in diameter may be used. These would be applicable simply at depressions where there are no permanent or well-defined ditches or streams, but not where the embankment would dam up the water in case of hard rains, which might by seeping through the earth cause settlement and possibly danger.

248. For any well-defined ditch or stream a culvert 2 feet square, increasing to 3 or 4 feet wide by 5 feet high, as the circumstances seem to require, will answer every purpose. If a larger water-way is required use the double box of corresponding dimensions, and if this is not sufficient the arch culvert should be used. The side walls vary in thickness from 2 to 3 feet according to height, and covering stones from 10 to 16 inches thick, according to the length of the span.

249. The vitrified terra-cotta pipes used are what are known as double-strength glazed pipe. They are now comparatively cheap, costing at most from twenty cents to one dollar per foot, according to size, and easy to handle; but there is always more or less danger of cracking or breaking, and for this reason, if no other, masonry culverts are preferred. These cost from two to five dollars per foot of length, depending upon convenience of material and kind of work required.

250. Cast-iron pipes have been used extensively in some sections of the country in diameters as great as 4 feet and in lengths that may be required. These cost from one to eight dollars per lineal foot, according to their weight. Special care is required in laying these pipes to prevent undue settling.

Article XXIV.

ARCH CULVERTS.

251. When a larger water-way is required than the limit of box culverts, say 4 feet span, arch culverts are constructed. These differ only in the size from ordinary arches, which have been fully discussed, except that the spaces between the abutments are generally paved, and an apron wall is built from side to side at the ends of the arch of a depth somewhat greater than the foundation bed of the side walls or abutments, the paving also extending above and below between the wing walls.

252. The usual manner of connecting the wing walls with the head walls is to throw them well back from the arch ring and then build them to the proper height with the usual batter,

and thickness proportioned as in retaining-walls, the angle of the wing being determined by circumstances—generally not more than 30 degrees with the axis of the arch. The only objection to these are the square shoulders presented at the entrance of the arch, forming a lodgment for ice and drift. This can be avoided to some extent by starting the wing wall at the front of the abutment, and carrying it up vertically to the springing line, and then commence the batter wall either at the springing line of the soffit, or leaving an offset at the springing line equal to the thickness of the arch ring and commencing the batter wall at the back or extrados of the arch ring. This was the plan adopted in the Philadelphia extension of the B. & O. Ry., except that the wing wall to the height of the springing line was carried up on a warped or twisted surface, that is, vertical where it adjoined the abutment, but assuming a gradually increasing batter to the end of the wing. This presents no shoulder to the height of the springing line, but presents shoulders above that line. It allows the wing wall to be bonded for its entire height into the head wall.

253. For any very small arch from 5 to 15 feet span they are generally full-centre circular arches. For spans from 20 to 30 feet the segmental arches offer some advantages: for the same length of intrados give a little longer span and the area of the water-way is a little greater, and for the same length of span there is a little less masonry, and does not require so great a rise, which is often an important consideration. On the contrary, the segmental arch requires greater thickness of arch ring and abutments.

254. Sometimes the wing walls are perpendicular to the axis of the arch, or in other words a simple extension of the head walls in a straight line; these offer no advantages, and are not used in any but very small arches.

255. Formula for the thickness of the arch ring have already been given. An arch ring somewhat thicker, both for circular and segmental arches, is advisable; the cost will not be much, if any, more, and presents a better and stronger appearance.

256. Trautwine gives the following rule for determining the thickness of abutments for arches in feet at the springing line, for any abutment the height of which does not exceed $1\frac{1}{2}$ times the thickness at the base. Thickness of abutment at springing equal to

$$\frac{\text{radius in feet}}{5} + \frac{\text{rise in feet}}{10} + 2 \text{ feet.}$$

The radius in this formula is that of a circle passing through the two springing lines and the crown, on the soffit. As it is always practicable to find a circumference passing through any three points in the same plane, this formula is applicable to a semicircular, a segmental, or elliptical arch. Laying off this distance and the height of the wall perpendicular to it, and at the bottom of this line drawing another line from one half to two thirds of the height, the line closing this quadrilateral would represent the surface of the back of the wall. This thickness is given to resist the thrust on the wall arising from the earth pressure of the embankment of any height over and around the wall. This does not take into consideration the thrust on the abutment by the arch, which is in the opposite direction to the earth pressure and at least neutralizes it in part, nor any support arising from the wing walls. This will no doubt give sufficient thickness in any case, but to resist the effects of the rolling load it is supposed that the earth has been deposited behind the abutments.

257. Applying this formula to spans 6′, 10′, 16′, with a rise of one sixth of the span, segmental arch, in the first case the radius equal to 5, in the second equal to 8.2, and in the third equal to 13.2.

The rise in feet would be, respectively, = 1, in the second = 1.7, and in the third = 2.7. Thickness (Trautwine's formula) of abutment at top in feet = 3.1, in the second = 3.8, and in the third = 4.92. Actual thickness of abutment at top in feet in same cases = 3.3, in the second = 4.6, and in the third = 5.6.

Actual depth of key-stone at crown = 1.0 foot, in the second

= 1.3 feet, and in the third = 1.6 feet. For semicircular spans, as above, actual depth of key-stone at the crown = 0.60 foot, in the second = 1.0 foot, and in the third = 1.2 feet. Actual thickness of abutment at top = 3.0 feet, in the second = 3.6 feet, and in the third = 4.6 feet.

Trautwine's results agree very closely with those in actual use for a semicircular or full centre arch, but smaller than the practice for segmental or elliptical arches. The practical rule is to add one third part additional in these cases, but this would be considerably in excess of the actual thickness of abutments given in the table above.

258. But, as stated before, the best rule is to extend the line of pressure from the top to the bottom of the abutment, and if this line remains in the middle third of its thickness, it will be stable so far as the thrust of the arch is concerned, and determine its stability against the earth pressure by the empirical rules for the thickness of retaining-walls; the least thickness thus determined will certainly be ample, as the two pressures balance each other in part.

259. But the earth pressure is hard even to approximate, as evidently the condition would be that of a surcharged wall; for by removing the arch ring and the earth above it the abutment would simply be a foot wall at the bottom of a mass of earth sloping away from the top of the wall at the angle of repose, either resulting from the removal of the earth to that slope or allowing it to assume its natural slope. At the solution of this the wonderful penetration of Mr. Rankine seems to falter, but he suggests the following:

$$\frac{t''}{x} = \cos\phi \sqrt{\frac{w'}{6qw} \times \frac{x}{1 + \sin\phi} + 2c} \div (x + 2c),$$

in which t'' = the thickness of the surcharged wall, c = height of surcharge, x = height of wall, ϕ = angle of repose of material, w' = weight of a cubic foot of the earth, w = weight of a cubic foot of the masonry in the wall. Mr. Trautwine gives the formula as follows: True theoretical thickness = weight of earth $\times$ 0.643, in which the weight of the earth is that of a triangular prism, whose base is formed by the height of the wall, the prolongation of the plane of maximum pressure to its intersection with the slope of the ground, thence by the length of this slope to the top of the wall. The moment of this equated to the moment of the weight of the wall will enable us to determine the thickness as in retaining-walls.

260. With a surcharge two times the height of the wall, he gives the following as safe thickness: cut stone 0.58, rubble or brick 0.63, of the height of the wall; and for a surcharge nire times the height of the wall, cut stone 0.65, rubble or brick 0.80, of the height of the wall. These thicknesses are roughly one and one half times of those when the earth is level with the top of the wall.

261. Whether these are right or wrong, they have but little bearing in the case of the abutments of the arch; for if the earth, as is often the case in railroad abutments, and which is the general case for culverts under highway bridges, only extends a few feet above the top of the arch, it evidently would be considered safe to consider the half arch, with its abutment and weight above, as the equivalent of a vertical-faced wall of the height of the embankment, and find the thickness of the wall to insure stability, as in retaining-walls, or using the thickness followed in practice, that is, from two fifths to one half the height. Take, for example, a semi-circular arch of 20 feet span, the rise 10 feet + arch ring + spandrel = say to 15 feet, and height of abutment between footing-courses and springing line 10 feet, the equivalent vertical-faced wall would then be 25 feet, and two fifths of this would be 10 feet = required thickness, or one half the height would be $12\frac{1}{2}$ feet. This would evidently be ample. Mr. Trautwine's rule for the thickness of abutments would give 5 feet at top and 6.6 at base.

262. The writer does not believe that any greater height of embankment, no matter how high, would require any greater thickness of abutment wall, and even doubts if the pressure would be any greater; but if it is, the greater stability of the wall, resulting from increase of weight of material above, would at least balance the increased thrust. The abutment would only give way by sliding or crushing, as the arch as a whole would be stable against overturning on account of the balanced pressure from the two sides.

263. It is not uncommon to see little foot walls supporting as it were sloping embankments ten times their own height, even when the material would continue to cave and slide down. It is true that these walls are always made of thicknesses equal to their heights, or even greater.

264. In these calculations neither the adhesion of the material, nor the support that the earth gives to itself by a tendency to be self-supporting by arching itself above the arch, nor the adhesion or cohesion of the mortar in the masonry, is considered.

265. If the above reasoning is not true, the linings of tunnels would be ridiculously thin, being only a few bricks in thickness, say from two to three feet, or from 5 to 8 rings of brick laid flatwise. It is true that such linings would not hold the pressure if it should once take a move, but are ample to prevent any movement. Conditions are of course different under a bank of loose earth thrown up, even after the lapse of time.

ARTICLE XXV.

THE COST OF WORK.

266. IT is considered by some useless to give the cost of any particular kind of work, as this depends upon so many contingencies and these varying rapidly from year to year, depending upon the abundance or scarcity of materials, the amount of work being carried on, the character of the specifications, the scarcity or abundance of labor, the time in which the work is to be completed, as well as the amount of work to be done at any one time and place: all of these things render anything but an approximation to the cost uncertain. It, however, serves as a guide. It can be stated as a general principle, that it will generally prove economical in the end to pay fair prices to responsible parties, and not to endeavor to get work done at a price less than it is really worth, and thereby secure only irresponsible contractors, who will not only execute the work badly, will never execute it in the specified time, will give all kinds of trouble, will endeavor to impose on you by numerous extra bills, by enormous charges for extra work, but too often, after performing that part of the work in which the greatest profit exists, abandon the work, leaving large unpaid bills for material and labor to be paid by the company, and a larger cost to complete the remaining work, than would have been really required to do the entire work, if let to capable and responsible men. Almost all important work is done by contract, because it is the cheaper in the long-run.

267. Ordinary brick or stone masonry, concrete and earthwork, are generally paid for by the cubic yard. Brickwork for the walls of houses either by the cubic yard or by the perch or face measure, for walls of definite thickness, say one and one half brick or 12 inches; if the wall is two times one and one half bricks, the area of the surface is doubled; if only one brick thick, then two thirds of the surface is taken. All openings are sometimes included, sometimes omitted, and sometimes averaged—either according to custom or agreement, or sometimes by the 1000 brick, allowing so many to the cubic yard—about 500. The best dressed work, such as coping, cutwaters, or raising stones, is paid for either by the cubic yard or the cubic foot. Frame timber is generally paid for by the 1000 feet B.M.; B.M. meaning board measure. The unit being 1 foot B.M., that is, a plank 12 inches long, 12 inches wide, and 1 inch thick, a stick of timber 15 feet long, 12 inches broad, and 10 inches thick would then contain 150 feet B.M. Cross-ties are paid for at so much apiece. Piles are generally paid for by the lineal foot, either for the ordered length, or for that portion left in the work, and allowing so much for the cut-off in addition.

268. Trestle-work is sometimes paid for at so much the running foot of a completed trestle. This, in many respects, is the most satisfactory mode of estimating. Iron, whether as bolts, rods, cylinders, and screw-piles, so much by the pound, and, in case of screw-piles and cylinders, so much for each foot sunk below water or the bed of the river.

269. In sinking caissons, so much per cubic foot of material moved, estimated by multiplying the bottom area by the distance from the surface of the water or the bed of the river to the lowest point reached by the caisson, or the lowest point of the foundation bed.

270. Again, all of these separate items on any work may be paid for in a lump sum. The contractor is likely to get the best of this, as he will be pretty sure to make a liberal allowance for contingencies; and if the estimate is anyways doubtful, it is a great temptation to do poor work, and if any loss occurs, the company will in general make it good, thereby making the work cost more than was anticipated.

271. A responsible party who gives a reasonable bid is in general the safest to accept; the highest bidders do not want the work very much, and the lowest want it too much.

272. Stone-cutting is paid for sometimes by the actual number of square feet cut, or simply by the face measurement of the stone; the finest dressing by the day, or by the actual surface cut.

273. However the work may be done, the contract should be clear and distinct, and full as to all essential conditions and requirements, avoiding at the same time useless and onerous conditions in regard to minor details, which are never carried out, and only give excuse for adding a good percentage to the profit arising legitimately from the work. Say in the specifications what you mean and mean what you say.

274. Contractors are generally required to furnish all materials—tools, derricks, engines, boilers, and everything, called the plant—necessary to carry on the work properly and expeditiously. Poor contractors will furnish broken-down carts and mules, worn-out boilers, engines, and pumps, barrows and tools in dilapidated condition, and all often in insufficient quantities, causing loss of money and time to both contractor and company. The company should always reserve the right to supply the deficiencies, if any exist, at the contractor's expense.

ARTICLE XXVI.

THE COST OF MASONRY AND CONCRETE.

275. THE following is the actual cost of masonry, concrete, etc., in some important works in the writer's experience:

	Per Cubic Yard.
Granite piers, not including cement, 13,767 cubic yards	$13 00
" " not including cement, second-class masonry	10 00
" pedestals, not including cement, first-class masonry	13 00
Limestone piers, first-class masonry, not including cement, 5652 cubic yards	12 00
Limestone abutments, second-class masonry	8 00
" coping	15 00
Granite arch stone	16 00
Brick in walls of houses, including mortar, $11.00 per 1000	7 00
Brick arch ring	9 00
Sandstone piers, cement included, about 2000 cubic yards	11 00
" " " " " 10,000 " "	14 30
Retaining-wall, rubble, not including cement	4 50
Brick wall piers, 2000 cubic yards	15 00
Concrete in cribs, not including cement, 18,146 cubic yards	6 00
" in interior of caissons, not including cement, 6036 cubic yards	15 00
" in " " " " " " 1500 " "	13 00
Box-culvert masonry	4 50
Paving	2 75

In the above table the cost of granite and limestone and sandstone piers included all coping and cutwater stone, and also the cement in the sandstone piers; in the granite and limestone piers the cement was furnished by the railroad company. The sandstone in large part was hauled over 100 miles by rail. Sandstone piers in the same bridge built of local stone, with a fair profit to the contractor, only cost $10.00 per cubic yard. $12.00, instead of $14.30, would have been a good price for the work, and could have been done at that figure; but the president of the road preferred one contractor at $14.30 to another equally as good, in the writer's opinion, at $12.00. In the

25,682 cubic yards of concrete as above there was used about 24,230 barrels cement; the average cost of this was $2.08 per barrel. About two thirds of this was Portland cement, of Alsens, London, and Saylor's American brand; these cost, respectively, $2.75, $2.80, and $2.50 per barrel. The Hoffman and Norton Rosendale cost about $1.29, delivered on the work.

In the above table the price of brick in piers costing $15.00 per cubic yard was caused by the fact that brick were very scarce and difficult to get, and had to be transported on barges for a long distance up a very rapid river, and could only be transported during the rises in the water. Under ordinary circumstances $8.00 per cubic yard would have been ample. The box culvert masonry is higher than usual, owing to small quantities and long haul. The paving should not usually cost more than $2.00, but good paving, which is set edgewise and properly laid, with the care necessary to avoid any undue deflection or obstruction to the current, is worth a fair price, as to some extent it may have to act as an inverted arch. It is well to fill the interstices between the stones with gravel, chips of stone, or even sand, as it will prevent undermining. The flow of the stream itself is likely to do this filling during the rises in the stream. Careless paving is worth but little, and owing to its supposed want of importance is often hardly worthy of the name. It should be at least from 10 to 12 inches thick, and should be laid slightly concave on top, the edges a little higher than the centre. It is well also for this paving to extend well beyond the apron walls, or even to the end of the wings. It should never be placed under the abutment or wing walls, but between and abutting against them well above the bed.

COST OF QUARRYING, DRESSING, LAYING, FINISHING, AND ALL TOOLS, MACHINERY, AND MATERIALS.

276. Cost of quarrying varies greatly with the kind of stone, the condition of the quarry, cost of labor, distance of quarry, but may be roughly estimated.

	Granite.	Sandstone.
Good size stone for rubble..........	$1.00 per cu. yd.;	$0.50
Good size ashlar stone.............	$4.00 " "	$3.00
Dressing beds and joints...........	$0.30 " sq. ft.;	$0.15
Cement and sand..................	$1.00 to $0.50 per cu. yd.;	$1.00 to $0.50
Laying	$2.50 to $1.75 " "	$2.50 to $1.75
Hauling depending on distance......	$1.00 to $2.00 " "	$1.00 to $2.00

At the above rate a cubic yard of face or dressed stone would cost in the pier for granite $18.00 per yard, assuming 35 square feet of dressed surface. The backing would cost the same as the above, less the cutting, but allowing $1.00 for hammering and $1.00 for the mortar, or $8.50. In a pier containing 1500 cubic yards, one third would be cut stone and two thirds backing. Actual cost per cubic yard would be $11.60, provided the cement was furnished by the company, which is a good plan in many respects, but is apt to lead to a liberal use of cement; but the satisfaction is that you will insure better cement and better work.

277. The writer in this calculation assumed a stone 6 ft. $\times$ $2\frac{1}{4}$ ft. $\times$ 2 ft. just making one cubic yard, and assumed the beds and joints to be cut true throughout, the face left rough. Stones of this size are rarely of such perfect shape. The joints of the stones are not cut true more than from 1 to $1\frac{1}{2}$ feet from the surface. The upper bed is not required to be cut with the same nicety and care as the lower bed, and it is far better to be rigid in regard to the lower bed and allow if necessary the upper bed to be a little rougher towards the back or the tail of the stone, for by this means you insure solid work and avoid to a large extent cheating, consequently bad work somewhere in the structure; in this way you may save three or four square feet of dressing, which is equivalent to from 90 cents to $1.20 per cubic yard, or the actual cost of the work will not exceed $10.50 per cubic yard, and for $12.60 should allow 20 per cent profit to the contractor. Good first-class granite piers can be built for at most $13.00 per cubic yard. The cost of first-class sandstone ashlar would be on the same basis of calculation $9.50 per cubic yard. The actual cost would be \$7.90 and allowing 20 per cent profit to contractor, or \$9.48 per cubic yard of masonry in the pier. Limestone masonry would not materially differ in cost from that of sandstone.

278. Good brick-work in walls of buildings can be constructed for \$7.00 per cubic yard, and from \$10.00 to \$13.00 per 1000. In tunnels, from \$8.00 to \$9.00. First-class masonry, \$10.00 to \$12.00. Second-class masonry or brick-work for arches, \$8.50. Box-culvert masonry, from \$2.00 to \$5.00. Concrete from \$4.00 to \$8.00, varying largely in proportion to the kind of cement used and proportions of sand and cement, and nature of the broken stone used. Rubble, from \$3.50 to \$5.00. Paving, from \$1.00 to \$2.00. Sand will cost, according to quality and length of haul, quantity, etc., from 20 cents to \$1.00. Cement, ordinary, in barrels, from \$1.00 to \$1.25 per barrel; in bags, about 10 cents less, say 90 cents to \$1.15. Portland cement, from \$2.00 to \$3.00 per barrel. Brick cost from \$6.00 to \$8.00 per 1000 brick, according to quality and demand.

Article XXVII.

DIMENSIONS OF PIERS.

279. The following are some of the dimensions and forms of piers and abutments constructed by the writer:

The Susquehanna River bridge was about 6200 feet long, arranged as follows: 2 spans, 1 through and 1 deck, 520 feet; 4 deck spans 480 feet, 2 through spans 375 feet, 1 deck span 200 feet. There were eleven piers and two abutments. Six of these piers were in water and five on land. Of those in water five rested on pneumatic caissons sunk from 60 to 90 feet below the water surface; one built inside a coffer-dam; all founded on rock, except two, which rested on beds of large bowlders mixed with gravel and sand about 70 feet below water surface. The masonry commenced on the crib at varying depths below the water surface, and was built up in steps or offsets to a point about 4 feet below low-water, at which level the neat work commenced, and was carried up to the proper heights above high-water, which for the piers carrying the 520-foot spans were 90 feet, and for the others or deck spans the tops of the piers were lower by the depth of the truss, from 40 to 50 feet. The piers were generally under coping 32 feet long and 10 feet wide for the low piers, and 35 feet long and 11 feet wide for the four high piers carrying the through spans. The batter was $\frac{1}{2}$ inch to the vertical foot from the top to the footing-courses on both sides and lower end. On the upper end, about 12 to 15 feet above low-water, a cutwater commenced, sloping downwards at an angle of 45 degrees, so that the cross-section of the pier at the top of the footing-courses would be about 20 feet by 61 feet long, to which the offsets would add about 10 feet all round. With the exception of the upper end of these piers to the top of the cutwater, these piers were square-ended from the bottom to the top. The cutwater was finished with a blunt triangular end. The coping and the triangular ends were cut to a smooth surface; the other parts of the piers were first-class ashlar masonry, rock face, with pitch line on the joints or edges of the stones. All of these piers had large raising stones on top of the coping 6 ft. × 6 ft. × 22 in. (Figs. 49 and 49 (*a*).)

280. There was in addition about 2300 linear feet of iron viaduct divided into 30-foot spans requiring about 154 pedestals reaching only a few feet above the surface of the ground. The pedestals were $3\frac{1}{2}$ feet square under coping; coping-stone 4 feet square and 15 inches thick, projecting 3 inches over shaft, the trestle being from 40 to 60 feet high.

281. The total cost of this bridge was $1,737,266, as follows: Foundations, $469,066; masonry, $208,000; superstructure, $1,060,200; and it was completed in two years from the time of letting contract. All things considered, it can be considered as executed both economically and expeditiously. All masonry was constructed of granite obtained from quarries a few miles above the site of the bridge. The raising stones were brought from near Wilmington, Del., as stones of the size required could not be obtained from the other quarry; and in addition the Port Deposit granite had certain seams crossing the natural beds, which rendered it uncertain for such heavy concentrated loads. The total quantities of materials used in this bridge, exclusive of superstructure, were as follows:

282. TABLE OF QUANTITIES AND COSTS.

Timber in caissons, cribs, and coffer-dams	2,727,755 ft. B.M.	@ $46.80 =	$127,658.93
Iron in caissons, cribs, and coffer-dams:			
Screw bolts	124,306 lbs.	0.06	7,458.36
Drift "	216,028 "	0.05	10,801.40
Spikes	44,650 "	0.05½	2,455.75
Cast washers	15,206 "	0.02⅛	323.13
Total concrete in air-chamber and in excavations below cutting edge	4,036 cu. yds.	15.00	60,537.45
Total concrete in cribs and under piers	11,141 "	6.00	66,846.00
Excavation, sinking caisson to cutting edge	781,934 cu. ft.	0.20	156,386 80
Excavation, sinking caisson below cutting edge	34,452 "	0.20	6,890.40
No. of bbls. Portland cement	10,620 bbls.	2.80	29,736.00
" " " Rosendale and Cumberland cement	3,668 "	1.29	4,731.72
Total for foundations			$473,825.94
" " Masonry, first-class	14,582.80 cu. yds.	13.00	$189,576.40
" pedestal masonry, first-class.	429.53 "	13.00	5,583.99
" for masonry, second-class	817.00 "	10.00	8,170.00
" rubble masonry and concrete.	1,314.61 "	6.00	7,887.66
" for substructure			$685,043.89
Coffer-dam for pier 5, estimated			5,000.00
Cement in masonry and concrete	8,700 bbls.	1.29	11,143.00
Cost of engineering, approximate			20,000.00
Extra bills handling material, extra work, etc., estimated			20,000.00
			$741,186.89

This includes some items not included in paragraph 281.

283. To determine the cost per cubic yard of the volume whose base is the area of the bottom of the caisson and whose height is the depth sunk, which is the most convenient form for arriving at an approximate estimate of the probable cost

of any proposed structure of this kind, we will take each caisson separately, as all the distances sunk differ, and also the dimensions of the caissons. We have then the following for the above structure :

	Dimensions at Bottom.	Area in sq. ft.	Depth in feet.	Volume in cu. ft.	Volume in cu. yds.	Cost per cu. yd.	Total Cost.
Caisson No. 2..	63.27×25.93		68.32	112,124	4153	$15.08	$62,613.41
" " 3..	67.27×25.93		70.72	123,402	4571	15.25	69,603.92
" " 4..	79.40×32.85		65.25 59.9	159,588	5911	15.03	88,830.64
" " 8..	70.85×32.61		88.4 76.00	189,578	7021	15.56	109,248.70
" " 9..	78.19×42.27		78.26 65.01	231,692	8581	16.17	138,769.42
					30237	15.50	469,066.09

The total cost in this table should agree exactly with the corresponding item in preceding table, viz., $473,825.94; but in the above table the concrete is calculated by averaging the cost of cement, and in addition there is some 200 yds. of concrete under one of the piers not included in the caissons proper. The iron is also taken at an average price in the above. The above table is, however, a close approximation to the actual costs. If the displacement is measured from the bed of the river and not from the water surface, the average cost per cubic yard on the above unit prices would be considerably greater than the above. As for example, in caisson No. 2 the displacement would be only 94,504 cu. ft. instead of 112,124 cu. ft., and 3500 cu. yds. instead of 4153 cu. yds., making the cost per cubic yard $17.89 instead of $15.08 per cubic yard. These would depend upon the terms of the contract. In this bridge the excavation or displacement was measured from the water surface. Mr. Baker in his work makes this $19.93 per cubic yard, and the average for the entire work $22.69, instead of $15.50, as in table. The above quantities and costs are taken from the writer's final estimates on the work. In caissons 4, 8, and 9 the actual depths to the bottom of the cutting edges are respectively, 59.9, 76.00, 65.01;

whereas to the lowest point of rock the depths are, respectively, 65.25, 88.4, and 78.26. In these caissons the cutting edges rested on rock at one end, and were, respectively, 5.35, 12.4, and 13.25 above rock at the other. This will again be referred to in discussing pneumatic caissons.

THE SCHUYLKILL RIVER BRIDGE, B. & O. RY.

284. This bridge, located near Gray's Ferry, Philadelphia, was comparatively short and low, requiring a drawbridge. There were two abutments, three piers, and one pivot pier. The spans were comparatively short, being as follows: One span 201 ft.; draw-span 242.64 end to end, 75 ft. clear opening at low-water; one span 200 ft., one span 152 ft. The line crossed the river at an angle of 53° 15′ with the direction of the current, requiring the piers and abutments to be very long in proportion to the length of the span. The east abutment, U-shaped in plan, was founded on rock only a few feet below the surface of the ground. Pier No. 5 was located on the edge of a rapidly dipping rock, and was built inside of a coffer-dam; the rock on the east side was exposed at low-water, and on the west side was from 10 to 15 ft. below the water surface. The range of the tide was from 5 to 10 ft. An ordinary coffer-dam was first tried, but owing to the great difference of the depth on the two sides of the dam, and the silty nature of the material overlaying the rock on one side, and no material on the other, this dam failed: a good crib-dam would have stood, but on such a sloping surface it would have been difficult to frame and handle. After the failure of the first dam, a contract was made with Mr. J. E. Roninson to put in his patent coffer-dam (this will be explained under Coffer-dams), and after much delay and many breaks we finally reached the rock. The remaining piers, 2, 3, and 4, rested on pneumatic caissons sunk to the rock. The west abutment, U-shape in plan, rested on a pneumatic caisson. The following is a table of quantities and costs:

TABLE OF QUANTITIES AND COSTS.

Caissons.	Dimensions in ft.	Depth Sunk in feet.	Excavation in Sinking Caissons.		Concrete		Caissons, Cribs, and Coffer-dams.		Piers and Abutments.			Vol. in cu. yds.	Total Cost.
			cu. ft.	cu. yds.	in Air Chamber, cu. yds.	in Crib, cu. yds.	Iron, lbs.	Timber ft. B.M.	Width, ft.	Height, ft.	Length ft.		
		Cost	20 cts.	$5.40	$13.00	$4.50	4 cts.	$40.00 per 1000				$8.00	
West abut., No. 1.	44×45	59.79	118404	4400	342	1447	56532	383896	6	24	35	640	$57374.98
		84.14				$6.00			15	<bottom>	50	$12.00	
2.	65.67×23.67	76.20	124310	4604	593	1610	47712	321946	8	33.63	43	651	64829.32
		58.74							diam.				
Octagonal 3.....	Diameter 47	52.78	92404	3422	676	1629	107400	366984	33	29.5		1135	69639.16
		44.39							37	<bottom>	37		
4.....	65.5×23.5	37.02	62647	2320	594	325	54033	375727	8	33.63	43	651	47203.80
	circular dam	16.17							15.33	<bottom>	50.33		
Coffer-dam 5....	Diameter 60	5.5							8	40	43	735	14820.00
												$8.00	
East abutment...									36	24	35	640	5120.00
								$35.00				$12.00	
Pivot guard pier.							44331	201304				1200	23218.80
Pile guard pier..	30 cts. per	lin. ft.	20.000	lin. ft.									6000.00
Cement.........	In caissons	$7489.	00. I	n mason	ry $28	26.00							10350.00
				14764								5652	$298521.06

285. In the above table the price for timber is taken at \$40.00 per 1000 B.M. This is an average of the prices. The actual contract price was \$45.00 for timber in caissons, \$38.00 in cribs and coffer-dams, and \$35.00 for timber in open caissons sunk on top of piles for the guard piers. The cost of iron is also an average for the different grades. Taking the four pneumatic caissons, exclusive of the masonry, the cost would be \$239,047 − \$34,364 + cement \$7489 = \$212,172.26. The total displacement in cubic yards is 14,764; we find that \$212,172.26 ÷ 14,764 = \$14.40, which is the cost per cubic yard under the water surface. This is generally the basis upon which the contract for piers is determined. In the above case if the excavation or displacement is taken from the mud line the cost would be about \$18.00 per cubic yard. It is always better to make the water surface the starting point, as in all cases the low-water surface can be definitely ascertained or agreed upon, marked, and preserved as a datum. The bed of the river may fill up or scour out, and the datum will be uncertain and cause confusion and uncertainty. The timber used in these caissons was entirely of yellow pine. The masonry was of a tough limestone, easily splitting in one direction, but very difficult to split in the other. The pivot pier was 33 ft. in diameter under coping, 35 ft. at bottom of shaft, two offset courses, and 38 ft. at bottom of offset courses. The coping was 16 in. thick. The other dimensions in the table are given under the coping and at the bottom of offset courses.

OHIO RIVER BRIDGE, POINT PLEASANT, W. VA.

286. From bank to bank on this river was 1370 feet, divided into five spans, respectively 250, 250, 250, 420, and 200 feet, by six piers, two on land and four in the water. For the land piers pits were dug about 15 feet deep, and piles driven in the bottom, 2½ feet centres, cut off about 1 foot above the bottom, then capped with 12 × 12 inch pine, the intervening spaces filled with concrete; this was then covered with a solid flooring of 12 × 12 inches, upon which the masonry commenced; concrete was piled up all around from the bottom to a point about 3 feet on the masonry. For the four river piers ordinary coffer-dams were constructed, and on the inside of these a timber crib was sunk. This work was prosecuted with great vigor and in the face of many difficulties, such as floods, intensely cold weather, and a suspension of the work for about six weeks during the lowest water and most favorable weather. The average depth excavated below the bed of the river was from 10 to 12 feet, through gravel and sand. The dams were strong enough to stand a rise of 15 feet, or a total pressure due from 25 or 30 feet. All of the foundations were completed, and in addition all the masonry of the piers, within 12 months from time of commencing, and a considerable number of the pedestals for the iron viaduct were also completed.

287. The iron viaduct was 2380 lineal feet, and of a height varying from 60 feet to 20 feet. The grade on the iron trestle was $1\frac{1}{2}$ feet in 100 on both sides of the river. The bridge itself was constructed on a 0.5 grade on the east and 0.25 on the west of the channel span. These piers were built entirely of sandstone, mainly from the Hocking Valley quarries, a hundred miles distant by rail, and partly from a local quarry, called Miller's Quarry. The following table gives the crushing strength of true cubes, 2 × 2 × 2 inches, using in the crushing

Location of Quarry.	Slight Signs of Yielding at Pressure per cube; per square inch.	Crushed or Split.		
		Pres. pr. cube in lbs.	Pres. pr. sq. in. in lbs.	
Hocking Valley, No. 1.	4,800 or 1,200	18,458	4,614	Without violence or noise.
" " " 2.	23,526 " 5,831$\frac{1}{2}$	27,885	6,971	Slight noise.
" " " 3.	6,650 " 1,662$\frac{1}{2}$	12,000	....	Not crushed.
" " " 4.	8,130 " 2,032$\frac{1}{2}$	15,930	3,982	
" " " 5.	17,620 " 4,405	17,620	4,405	Crushed suddenly without noise.
" " " 6.	18,740 " 4,685	18,740	4,685	No evidence of yielding whatever.
Miller's Quarry " 1.	....	14,942	3,735	
" " " 2.	....	15,442	3,860	

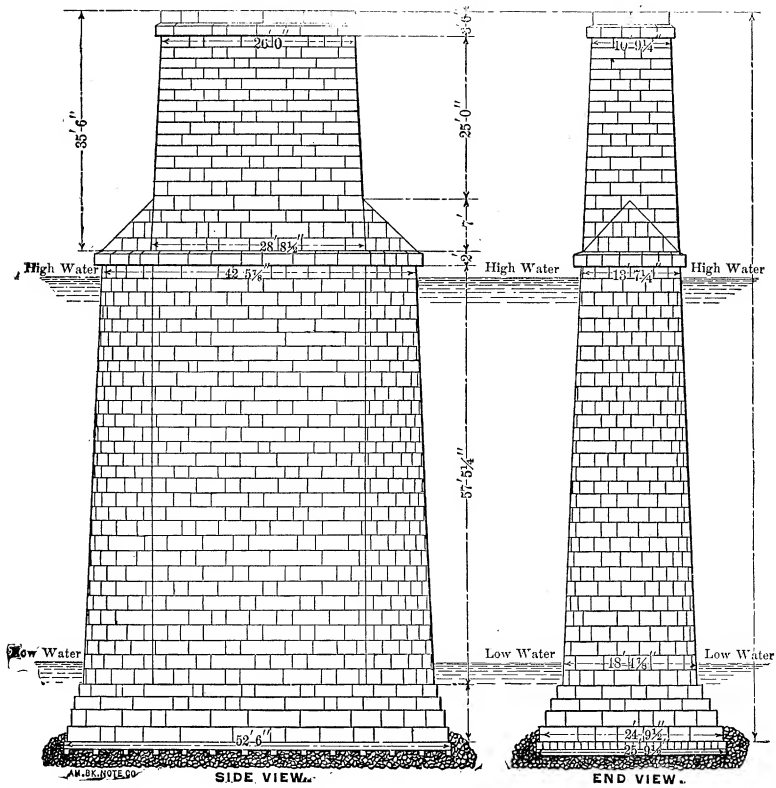
soft white-pine cushions. These cubes were dressed true in a marble yard.

The writer tested these specimens with the above results, and also many other specimens from various places; the general average was the same as above. The piers were built of the above stone. The tests were made at the Ohio State University, Columbus. These specimens were all comparatively fresh from the quarries, as we could not wait very long before deciding upon the quarry. The specimens would doubtlessly have resisted a higher pressure after seasoning thoroughly. Pasteboard cushions are now recommended as better than pine. The following are the quantities and costs:

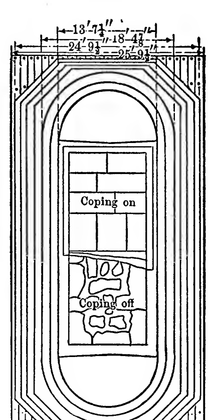
273,210 feet B.M. pine timber in coffer-dams, cribs, and foundations under piers.
244,412 " " oak " " " " main and sheet piling.
3,597 " " poplar " " " " sheet piling.
13,571 lineal feet of piles in foundations and coffer-dams.
3,499 cubic yards excavation sand and gravel.
649 " " concrete in foundations.
997 rip-rap stone.

Total cost of foundations	$64,652 62
Masonry in piers, first-class, 8,654 cubic yards, $14.30	123,756 92
Masonry in pedestals, " " 1,224 " " 14.30	15,912 00
Earth in approaches, 39,490 " " .22	8,687 80
" " pedestal foundations	2,304 02
371,962 feet B.M. in trestle approach, $30 per 1000 feet B.M.	11,158 86
11,965 wrought-iron screw and drift bolts, 5 cts	598 25
7,939 cast-iron washers and packing spools, 4 cts	317 56
Extra bills	174 64
Total cost of substructure and approaches, exclusive of iron viaduct	$227,562 67
Total cost of bridge proper, 1370 lineal feet, $126.50	173,213 00
" " " iron viaduct, 2380 " " 39.00	92,771 00
Total cost of completed structure	$493,546 67

288. The dimensions of piers and pedestals were as follows: Pier No. 1, carrying one end of one span 250 feet and end of iron viaduct, at top 22.22×6.5 feet, at bottom 25.54×10.04 feet, 43.84 feet high. Offset course 4.65 feet, height, and 28.53×12.53 feet at bottom, total height 48.49 feet,



(*a*) (*b*)



PLAN AND HORIZONTAL SECTIONS.

(*c*)

Fig. 17.—Pier No. 5, Ohio River Bridge, Point Pleasant, W. Va.

[*To face page* 135]

square ends. Piers 2 and 3, carrying 250-feet spans, top 23.00 × 9.00 feet, square ends, at 33.35 feet from top, 25.8 × 11.8 ft. Belt-course 2 feet thick, 38.92 × 13.22 feet, projecting 9 inches all around. The piers were lengthened at the belt-course by adding semicircular ends from that point to below low-water. Main wall under belt-course 37.42 × 11.72 feet, bottom of neat work 42.20 × 16.50 feet, height 59.39 feet. Then four offset courses 8.58 feet thick, bottom dimensions 49.00 × 23.00 feet. Total height 97.32 feet. Piers 4 and 5, carrying channel span 420 feet, top 26.25 × 10.55 feet, at top of belt-course 28.88 × 13.20 feet. Main wall under belt course 42.24 × 13.36 feet. Bottom neat work 46.92 × 18.04 feet. Three offset courses 5.6 feet high. Bottom dimensions 51.72 × 22.84 feet. Total height 96.86 feet for No. 4 and 101.60 feet for No. 5. Square ends to belt-course, rounded ends to bottom. Pier No. 6, carrying 200-feet span, top 22.00 × 6.00 feet, bottom of neat work 27 × 11 feet, height of shaft 59.88 feet. Two offset courses 5 feet thick, bottom dimensions 31.06 × 14.60 feet. Total height 64.88 feet. This case is entered into as well illustrating a good standard of dimensions and form of piers. Minimum dimensions for piers carrying length of spans above called for. All of these piers had raising stones of Berea, Ohio, sandstone, a hard, strong stone. The coping was doubled, the bottom course projecting 9 inches all around. Thickness of each course was 18 inches. A cone-shaped finish was placed at the ends on top of the belt-course in passing from the curved ends to the square ends. The pedestals were 4 feet square under coping, coping 15 inches thick, projecting 3 inches. Top of pedestal was from 2 to 4 feet above ground. Two offset courses below ground, generally built of only two stones to the course, sometimes three stones allowed in the footing-courses. It is best to arrange elevation of the top of the coping, where the ground will allow, so as to have as many trestle-bents of the same height as possible.

For elevation and plan of pier as just described, see Fig. 17, (*a*), (*b*), and (*c*), and Figs. 18 and 19.

ARTICLE XXVIII.

DEFINITIONS.

PARTS OF THE ARCH.

289. *Abutment.*—The masonry supports of the arch ring.

Skew-back.—A course of stone on top of the abutment, with an inclined surface from which the arch directly springs.

Arch Ring.—The masonry of the arch between the intrados and extrados.

Intrados or Soffit.—The under curved surface of the arch ring.

Extrados or Back.—The top curved surface of the arch ring.

Crown.—The highest part of the arch.

Springing Line.—The line on the soffit at the top of the abutment.

Haunches.—The lower part of the arch ring on both sides.

Spandrels.—A wall, or walls, built on the top of the arch, commonly one at each end and in the plane of the face, 2 to 5 feet high.

Span.—The horizontal distance between springing lines.

Rise.—The vertical distance from the springing to intrados at the crown.

Ring-stones.—A course of stones between two vertical planes; does not actually exist, as the stones break joints, there being no continuous joint in a plane parallel to the face of the arch. The face stones, or those seen on the ends of the arch, are frequently cut on top with a horizontal and vertical surface which project above the extrados.

String-course.—A course of stone extending from end to end of the arch.

String-course Joint.—The joints between the string-courses, continuous and in the same plane from end to end of arch.

Semicircular or Full Centre Arch.—One in which the intrados is a full half of the surface of a cylinder.

Segmental Arch.—One in which the intrados is less than the surface of a semi-cylinder.

Elliptical Arch.—One in which the intrados is part of an elliptic cylinder; one in which the rise is less than the half-span.

Pointed Arch.—One in which the rise is greater than the half-span, generally formed by the intersection of two equal circles. See Fig. 12.

DEFINITION OF PARTS OF PIERS, RETAINING-WALLS, ETC.

290. *Face.*—The exposed part of a pier or wall.

Facing Stone.—The stones that show on the face of the wall.

Backing.—The stones behind the facing stones in retaining-walls and between the face walls in piers, and well bonded to the face stones. Also called filling, whether of large stones, rubble, or concrete.

Batter.—The inclination of the face of a wall to a vertical, generally expressed in fractions of the height, as ½ inch, 1 inch, 1½ inch to 1 foot vertical; ordinarily ½ inch to 1 foot.

Bond.—The overlapping of the stones so as to tie the wall together. See Fig. 13.

Course.—A layer of stone between two horizontal planes or joints.

Joints.—The space between the stones, generally filled with mortar. The bed-joints are the top and the bottom, generally horizontal; and the side-joints, which are either vertical or inclined, generally vertical.

Stretcher.—A stone that shows its full length on the face and all stones parallel to it in the backing.

Header.—A stone that shows its end on the face and all parallel to it in the backing.

String or Belt Course.—A course of large stones, projecting from 6 to 9 inches from the face of the wall; generally dressed smooth on the exposed part, and also has a wash cut on it. Used mainly for appearances, and marks a change from a curved to a plain finish to a pier.

Coping.—A course of large stones on the top of the pier, projecting from 6 to 9 inches, dressed true on all surfaces; has a wash on the projecting part. Sometimes two coping-courses are used. The upper one has no projection, and in fact the lower coping projects beyond the upper 6 to 9 inches. These coping-stones are commonly bolted to the pier, or fastened to each other by cramps or dowels.

Pedestal or Raising Stones.—Large thick stones of some hard variety, placed on top of coping, upon which the ends of the bridge directly rest. These are not always used.

Pointing.—Cleaning out the joints to the depth of 1 to $1\frac{1}{2}$ inches, and refilling with good mortar.

Quoins or Corner-stones are stones at the corner showing header on one face and stretcher on the other.

Dowels.—A straight bar of stone or iron, fitting into holes cut in the sides or beds of adjacent stones so as to prevent one lifting without the other.

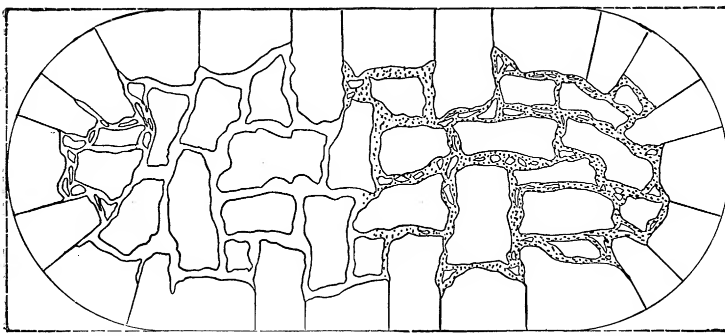
Cramps.—Iron bars 18 to 20 inches long, bent at right angles at the end for 2 to 3 inches and placed across the joint, the bent ends let into holes cut in the top of the stone; a groove also being cut between the holes so that the bar will not project above the surface of the stone. Hot lead, sulphur, or cement, is generally poured around the bar to fasten it in place.

291. The character of the masonry is determined by the size of the stone, the regularity of the courses, the amount of dressing or cutting.

Ashlar Masonry or Block-in-course.—Masonry laid in regular courses with bed-joints horizontal and side joints vertical; all the stones cut into regular blocks; all surfaces dressed smooth except the face. This may or may not be dressed.

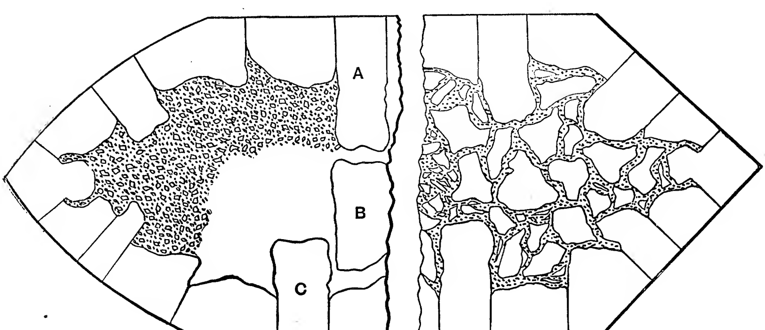
Random or Coursed Rubble.—Masonry in which the stones are cut to regular shapes, side joints vertical, bed-joints horizontal but not continuous, courses of varying thicknesses, stones being large and small, thick and thin.

Common Rubble.—Masonry in which the stones are built as



Ends Large Stone. Circular Backing.

FIG. 18



Pointed-end Concrete Backing. Triangular-end Rubble Backing.

FIG. 19.

[*To face page* 138.]

they come from the quarry, with no regular courses; the joints are not necessarily either vertical or horizontal. Large and small stones are used at random and with or without mortar.

Stones.—The upper and lower surfaces are called beds, the remaining parts are called sides, face, and back.

Quarry-faced.—When the face is left as it comes from the quarry.

Rock-faced.—When the face is roughly hammered so as not to project more than from 3 to 5 inches.

Pitch Line.—When a straight, well-defined line at the angles is cut all around the face of the stone.

Chisel Draft.—When a smooth, plane surface from 1 to $1\frac{1}{2}$ inch is cut around the face of the stone, forming well-defined and regular angles; the rest of the face left rough. See Figs. 17, 18, and 19.

292. The face of the stones may be left rough. If the face has no projection over $\frac{1}{2}$ to $\frac{3}{4}$ inch, it is said to be rough-pointed; if the projections are not over $\frac{1}{16}$ to $\frac{1}{8}$ inch, it is called fine-pointed, and is what is generally understood by dressed stone, whether for face or for beds; on the face it is made to look uniform and regular. When required to be of a smoother surface than the fine-pointed, it is generally said to be bush-hammered; this is, however, done by an instrument called the crandall, which consists of a number of double-pointed steel pins fastened close together in a slot at the end of an iron bar, and produce a smooth and more regular surface than the fine point. The bush-hammer is not commonly used by stonecutters. The crandall or bush-hammer is only required for dressing coping or cutwater stones. For perfectly smooth faces the stone is first sawed and then rubbed to a smooth surface. It is only used for ornamental purposes.

293. The common way of raising large heavy stones to their position on the wall is by means of derricks, which consist essentially of a mast of greater or less height, resting on a solid block of wood and a boom connected with it at or near the bottom, and also by a rope at the top. A hoisting rope passes from a drum or capstan over a sheave in the top of

the boom and thence downward, terminating in a chain which has hard steel hooks at the bottom. Holes, for the hooks, are cut into the sides of the stone a little above the line passing through its centre of gravity. This is the common mode. Sometimes a dovetailed mortise is cut into the top of the stone, thicker at the bottom than at the surface, and a lewis made of three pieces of iron, two of which are truncated wedges, the other rectangular. The wedge-shaped pieces are first inserted, and are forced apart by driving the straight piece between them. The hoisting chains are attached to the wedge-shaped pieces, which can not be pulled out without breaking the stone. This, however, is only applicable to hard, strong stone, unless the mortise is cut very deep. Another method is to drill two holes, in a plane passing through the centre of gravity of the stone, inclining towards each other at an angle of 90°, or 45° to the vertical on either side; strong iron bars are inserted in these holes, and chains are fastened to eyes on the other ends of the iron bars.

For details of common forms of derricks, see Fig. 20, (*a*), (*b*), and (*c*); and forms of derrick set on top of pier and lifted by screws as the masonry is built, see Fig. 20 (*A*).

TABLES OF ULTIMATE STRENGTH OF STONES, NATURAL AND ARTIFICIAL, TO RESIST CRUSHING, TEARING, OR CROSS-BREAKING, AS GIVEN BY SEVERAL AUTHORITIES, IN LBS. PER SQUARE INCH.

294. TABLE 1.

Resistance to Crushing, in lbs. per square inch.	Rankine.	Baker.	Trautwine.
Granite	12,861	12,000 to 21,000	6,222 to 12,444
Marble	....	8,000 to 20,000	4,000 to 9,340
Limestone	8,528 to 3,050	7,000 to 20,000	4,000 to 9,340
Sandstone	9,824 to 3,000	5,000 to 15,000	2,333 to 6,999
Slate	....	....	6,222 to 12,444
Brick	1,100	2,500 to 3,000	800 to 4,800
Brick-work	....	1,150 to 1,290	310 to 465
Brick-work in cement	800 to 1,000	1,650 to 1,850	465 to 1,162

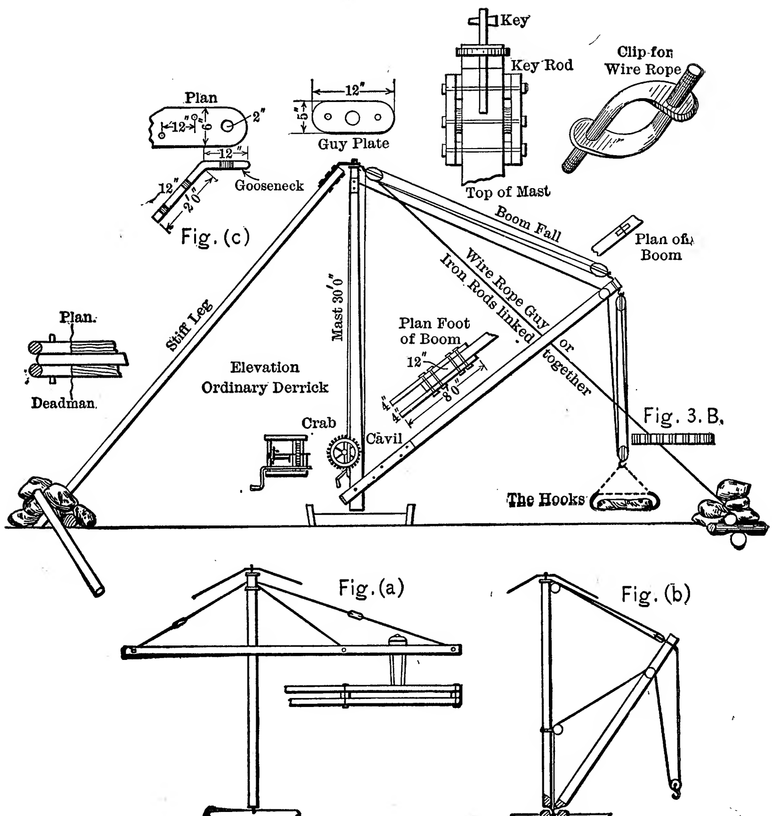


FIG 20.

[*To face page* 140.]

TABLE 2.

Transverse Strength or Resistance to Cross-breaking, in lbs. per square inch Modulus of Rupture.	Rankine.	Baker.	Trautwine.*
Granite	....	900 to 2,700	900 to 2,700
Marble	....	144 to 2,800	....
Limestone and Sandstone	....	576 to 2,340	360 to 1,260
Slate	....	1,800 to 9,000	3,600 to 5,700
Brick	....	269 to 1,796	180 to 540
Brick-work, common	....	....	....
Brick-work in cement	....	200 to 380	....

295. Although there are considerable differences between the resistance to crushing of the stones above given, no inconvenience or doubt need rise as to the strength of any of the above, as 200 lbs. per square inch is an unusual pressure, and this only exists under the largest and highest structures, and then only when the normal unit pressure is increased by wind pressure on the leeward side.

296. To apply the table of crushing strength to any structure, it is only necessary to multiply the unit pressure in the column by the area of the cross-section in the same unit to obtain the total resistance to crushing, as $R = pA$, in which p is the unit or coefficient of resistance to crushing in lbs. per square inch or per square foot, and A is the number of square inches or square feet in the base of the structure, and R the total resistance to crushing. For example, take the average area of a pier built of sandstone to be $22 \times 40 = 880$ square feet. In the column from Rankine's Engineering the least resistance of sandstone to crushing is 3000 lbs. per square inch $= 432{,}000$ lbs. per square foot, or 216 tons of 2000 lbs. per square foot, hence $R = pA = 216 \times 880 = 190{,}080$ tons. Now to determine the height of a sandstone pier that would crush at the base under its own weight (assuming that it does not give way by flexure or transverse strain, the limit

* Trautwine always takes the lengths of beams (l) in feet, in which case the moduli of rupture are only $\frac{1}{18}$ of the numbers in this table. See Trautwine, page 185.

of which is generally taken at a height not over 20 times its least dimension, a height which rarely occurs in practice. (The height of the Washington Monument is 500 feet to the bottom of the pyramidal finish, and the least diameter of the column $36\frac{1}{2}$ at top, and 50 ft. at bottom; the middle would be $43\frac{1}{4}$; therefore the height is only 14 times its least diameter.) The ultimate crushing strength is 216 tons per square foot, or 432,000 lbs.; assuming that the weight of sandstone masonry is 140 lbs. per cubic foot, we have $140 \times y = 432,000$, hence the required height $y = 3685$; and in the same manner the height of a brick column, assuming brick to weigh 125 lbs. per cubic foot, would vary from 600 to 900 ft. to crush of its own weight, and a granite pier about 8000 ft. high. In selecting the unit of resistance to crushing from the tables, whether you take the least or the average or the greatest, depends upon the kind or quality of the stone considered. (Table 1.)

297. The base of a brick chimney at Glasgow, Scotland, 468 feet high, bears 9 tons per square foot, and in high winds may have to bear as much as 15 tons per square foot, or 210 lbs. per square inch at base; and Mr. Trautwine expresses the opinion that first-rate hard brick laid in cement would carry without completely cracking 100 tons per square foot or 1400 lbs. per square inch.

298. So far as stone is concerned, the main application of the table of transverse strength is in the case of lintels over openings, which in general are to be considered as beams uniformly loaded; but as they may sometimes also be subjected to a single concentrated load at the centre, it will be well to apply the formula for both cases, although the first is two times as great as the second. The writer will use Rankine's formula, which is easy to remember, when the principle of moments is understood, is easy of application, and applies to all conditions of loading and supporting beams, which will be further explained in connection with timber. The formula is: $mWl = nfbh^2$, in which m is a factor depending upon the manner of loading and supporting the beam, W is the concentrated weight at the middle point between the supports; l is the length of

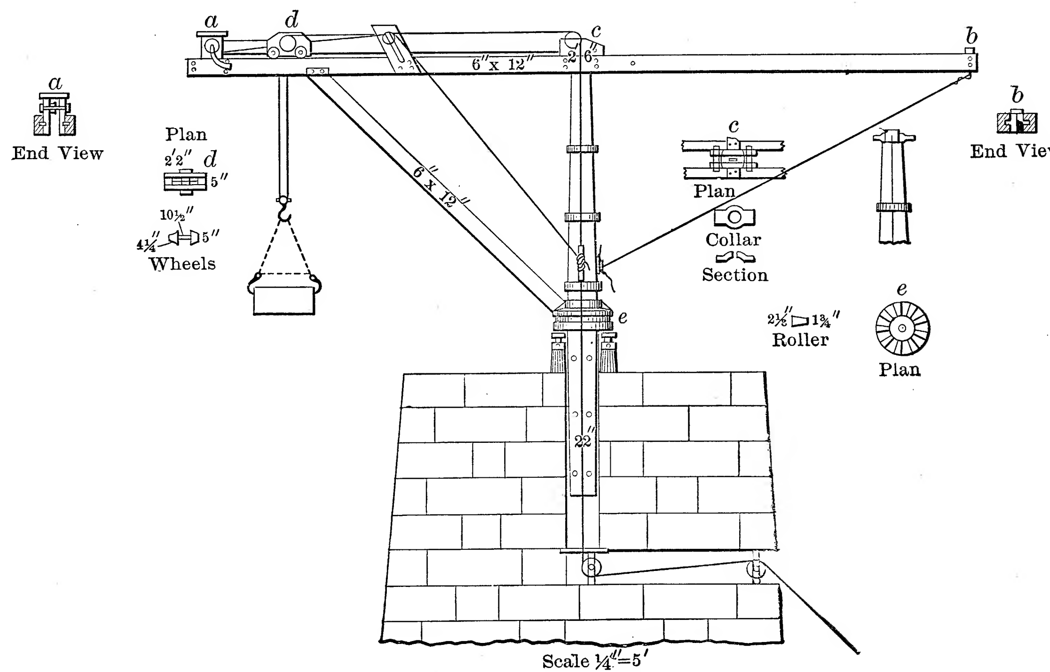


FIG. 20 (*A*)

[*To face page* 142.)

the beam or clear span in inches; n is a factor depending on the cross-section of the beam, and for rectangular beams is equal to $\frac{1}{6}$; f is the modulus of rupture in lbs. taken from the table; b is the breadth in inches, and h is the depth in inches. Suppose a lintel to be 10 inches thick, 2 feet wide, and 10 feet long, loaded with a single weight at the centre, the lintel to be rectangular in cross-section, and the stone granite; then $m = \frac{1}{4}$, $l = 120$ inches, $n = \frac{1}{6}$, $f = 2700$, as this would always be the best and strongest stone; $b = 24$ inches, and $h = 10$ inches. Substituting in the formula, we have

$$\tfrac{1}{4}W \times 120 = \tfrac{1}{6}\,2700 \times 24 \times 100; \quad W = \frac{4 \times 2700 \times 24 \times 100}{6 \times 120};$$

hence $W = 36{,}000$ lbs. centre-breaking load. When uniformly distributed over the beam, it would be double the above, or 72,000 lbs., as will be seen from the formula; W, in this case, $= wl$, in which $w =$ weight on a unit of length, and $m = \frac{1}{8}$, all other values of same; hence

$$\tfrac{1}{8}(wl)l = \tfrac{1}{6}\,2700 \times 24 \times 100; \quad \text{or,} \quad wl = \frac{8 \times 2700 \times 24 \times 100}{6 \times 120};$$

hence $W = wl = 72{,}000$ lbs. For other materials and other dimensions similar results can be obtained, using Table 2 for transverse strength.

299. In practice, under steady loads, it would not be safe to rely upon more than from $\frac{1}{4}$ to $\frac{1}{5}$ of the above loads; but it is safer to use only from $\frac{1}{8}$ to $\frac{1}{10}$ of the above results; that is, for a granite beam of the dimensions given above, it should not be loaded with more than 7200 lbs., or 720 lbs. per foot of length.

300. TABLE 3.

TABLE OF TENSILE STRENGTH OF MORTAR IN POUNDS PER SQUARE INCH.

(From BAKER.)

Hydraulic, with sand, 30 to 300, age from 1 week to 1 year, 1 sand, 1 cement.
Hydraulic. { neat } from 40 to 400.
Cement, { cement } from 100 to 800, age from 1 day to 1 year.
Cement and sand, from 80 to 350, age from 1 week to 1 year, 3 sand, 1 cement.

301. The adhesive strength of mortar varies greatly with the kind of cement used and the proportion of sand, the cleanness of the surface of the brick or stone, whether porous or not. Mr. Rankine gives 15 lbs. per square inch to limestone and 33 lbs. per square inch to brick. According to Baker, in Portland neat cement the adhesive strength varies for limestone from 57 to 78 lbs. per sq. in. and from 19 to 213 lbs. per square inch for brick; and when mixed, 1 cement, 2 sand, the adhesive strength varies from 5 to 140 lbs. per square inch, according to the character of cementing material and stone used.

302. The absorptive power may be taken as one part for from 80 to 700 parts in granite, and 1 part in from 30 to 60 of sandstone, limestone from 1 part in 20 to 1 part in 500, and for brick from 1 part in 4 to 1 part in 50, and mortars from 1 part in 2 to 1 part in 10. In general a small absorptive power is an indication of a good quality of stone.

303. According to Rankine, the expansion of stone is as follows: brick, .00355 of its dimensions; sandstone, .0009 to .0012; marble, .00065 to .0011; granite, .0008 to .0009 of their linear dimensions in a range of 180° Fahr.

304. TABLE 4.

TABLE OF WEIGHT IN POUNDS PER CUBIC FOOT OF MANY SUBSTANCES.

(From TRAUTWINE.)

Granite	168	Sand	99 to 117
Limestone	172	Sand, packed	101 to 119
Marble	172	Sand, wet	120 to 140
Sandstone	150	Clay, dry	63
Slate	175	Ordinary earth	72 to 92
Common brick	125	Ordinary earth, packed	90 to 100
Pressed brick	150	Mud	104 to 120
Masonry of—		Hydraulic cement	60 to 80
Granite	165	Portland cement	80 to 87
Rubble	125	Mortar, dry	100
Sandstone	144	Concrete	...
Common brick	125	Water	62.33
Pressed brick	140		

The above table is useful in determining the stability of retaining-walls, weight of structures, and force tending to overturn the wall or to cause sliding.

305. The following are the angles of repose or the angles of friction between different substances heretofore considered:

TABLE 5.

(From TRAUTWINE.)

		Coefficient of Friction.
Polished marble on polished marble	9° 6′	0.16
Polished marble on common brick	23° 45′	0.44
Common brick on common brick	32° 38′	0.64
Common brick on dressed soft limestone	33° 2′	0.65
Common brick on dressed hard limestone	31° 00′	0.60
Hard limestone on dressed hard limestone	20° 48′	0.38
Hard limestone on dressed soft limestone	33° 50′	0.67
Soft limestone on dressed hard limestone	33° 2′	0.65
Masonry and brick-work, dry	33° 2′	0.65
Masonry and brick-work mortar, damp	36° 30′	0.74
Masonry and brick-work, dry clay	27° 00′	0.51
Masonry and brick-work, moist	18° 15′	0.33
Wet clay	14° to 17°	0.25 to 0.31
Dry clay	21° to 37°	0.38 to 0.76
Damp clay	45° 00′	1.00
Shingle and gravel	35° to 48°	0.70 to 0.90

This table is useful in determining stability of walls and arches against sliding in connection with weight of walls and position of plane of rupture in calculating the thrust exerted against walls. To determine resistance to sliding of one body on another multiply normal component of weight of one body resting on another by the coefficient of friction, if the surfaces are inclined, and the entire weight if the surfaces are horizontal. What is the resistance of a block of dry masonry weighing 20 to sliding on any ioint? $20 \times .65 = 13$ tons.

306. TABLE 6.

THE BEARING POWER OF SOILS IN POUNDS PER SQUARE INCH AND TONS PER SQUARE FOOT.

	Rankine, Safe Load.		Baker, Safe Load.		Ultimate Load, tons per sq. ft.
	Lbs. per sq. in.	Tons per sq. ft.	Lbs. per sq. in.	Tons per sq. ft.	
Clay, dry....................	17 to 23	$1\frac{1}{4}$ to $1\frac{3}{4}$	55 to 86	4 to 8	15
Sand....	" "	" "	55 to 86	4 to 6	15
Clay and sand..............	" "	" "	55 to 86	4 to 6	....
Sand and gravel	" "	" "	111 to 140	8 to 10	....
Clay, wet	....	....	20 to 30	$1\frac{1}{2}$ to 2	....
Layer of clay over quicksand	....	....	20 to 40	$1\frac{1}{2}$ to $2\frac{1}{2}$	....
Alluvial soil, New Orleans...	....	....	7 to 14	$\frac{1}{2}$ to 1	...

We may then safely conclude that ordinary soils can be easily loaded with from 2 to 3 tons per square foot or from 4000 to 6000 lbs., and for softer soils, or firm soils resting on softer soils, 2000 to 4000 lbs. per square foot.

PART SECOND.

ARTICLE XXIX.

TIMBER FOUNDATIONS.

UNDER this heading are included simple timber foundations; piles, whether cut off under ground or under water, as well as when left standing above the surface, as is the case in pile trestles; framed trestles; timber piers for bridges; timber cribs, whether filled with concrete or broken stone; open timber caissons; coffer-dams; Cushing cylinder piers; etc.

TIMBER.

1. Timber is used extensively in the above structures for the following reasons: 1st. As a matter of economy. It is often impossible to procure stone or brick in any reasonable time or cost, but timber of some kind can commonly be procured which will at least do for a structure of a temporary character; and if under water or in wet or even constantly moist ground, it can be relied upon for the foundations of permanent structures, as it will not rot when constantly wet. When immersed in sea-water it is rapidly honeycombed and destroyed by sea-worms, unless creosoted. 2d. Timber which, either on account of its small dimensions or excess of sap, would be unsuitable for structures above ground, may be suitable for those under water or under ground. 3d. Timber is easily framed and handled, and can be transported overland or floated in large rafts on

rivers or streams, yet has the strength to bear heavy loads and strains. But be sure that the timber will always remain wet.

2. Under walls of houses pieces of plank, 5 to 6 feet long and from $2\frac{1}{2}$ in. to 3 in. thick, can be placed side by side on soft materials, thereby securing increased bearing surface, and by using from two to four courses placed at right angles to each other the base can be spread to a width of 10 to 12 feet, allowing structures of considerable weight to be built on very soft foundation-beds, such as silt or quicksand. Sometimes a series of rough logs are laid longitudinally, either side by side or at short intervals, the intervening space filled with sand, broken stone, or concrete, and one or more courses of plank placed over and at right angles to these; or two or more courses of logs crossing each other, the intervening spaces, if any, filled as stated above. This last constituted the foundation of the New Orleans Custom-house, a large, heavy, and massive granite building; it is true that in this some settlement has taken place, but no serious damage has resulted. By either of the above methods many houses, culverts, and other structures are safely and economically constructed. In such cases the probability is that whatever settlement takes place will be uniform under the entire structure, and no damage to the structure will follow unless high and heavy towers or steeples are bonded into the structure; in such cases the spread of the base should be such as to insure that the unit pressure shall be the same as under any other portion of the structure. Piles are better under such very heavy loads, and should always be used if there is any possibility of the material being washed or scoured out.

3. Under large and heavy piers for bridges it is not unusual to build cribs made of round logs or square timber, crossing each other and bolted together at each intersection, leaving cells or pockets to be filled with broken stone or concrete; in such cases the crib is really intended to confine the filling material, but of course supporting its proportionate share of the load. If the filling is gravel or broken stone, iron rods should be used to tie the sides of the crib together so as to prevent

any tendency to bulging; this is not necessary when concrete is used. The dimensions of such cribs should be from 4 to 6 feet greater all around than the masonry structures resting upon them. If the crib has to be sunk through any depth of water, a plank bottom will have to be used over the entire bottom, or at any rate under a sufficient number of the pockets to hold the weight necessary to overcome the buoyancy of the water. It is not advisable to endeavor to sink the cribs by building the masonry, as the cribs will rarely rest on the bottom in a perfectly level position. With broken stone or concrete filling this is a matter of little consequence, unless very much inclined, as it can be easily levelled with broken stone or concrete, and the masonry commenced by the use of a cofferdam if necessary, but usually the crib is built to within 2 or 4 feet of the surface of the water. The bed of the stream is generally levelled by dredging; this serves also to remove the soft and loose material at the bottom, but it generally requires removing the material over a large surface, if it is desirable to reach any great depth below the bed of the river, adding materially to the cost; and unless stiff clay is close to the bottom the obstruction to the current will almost always cause a scouring action, endangering the safety of the structure. It can hardly be recommended as a safe and satisfactory foundation. (See Elevations, Fig. 23, (*a*), (*b*), and (*c*).)

4. Many examples, however, exist, and have stood the test of time. The Parkersburg (W. Va.) bridge across the Ohio River was thus constructed; the piers of this bridge stand 90 feet above water, and rest on a bed of gravel and sand at a depth of 12 ft. below the bed of the river; the excavated pit was 100 by 50 ft.; the crib or grillage was composed of three courses of timber 12 in. by 12 in., bolted together, 78 ft. long by 28 ft. wide; this carried a pier 120 ft. high, with spans of 350 ft. Giving a pressure of 9080 lbs. or $4\frac{1}{2}$ tons (about) to the square foot on gravel and sand. A rod was driven 25 ft. into this material. An open caisson was built, the grillage forming the bottom, this was sunk on the gravel bed by the weight of the masonry itself, and was practically level when it rested on the

gravel. These piers were finished 15 ft. wide on top; 10 ft. would have been ample.

5. If a crib is to be sunk on a bed of rock which is very irregular or much inclined, two methods of procedure are open:

1st. To blast the rock to a level or nearly level surface; this is difficult, slow, and expensive.

2d. If the rock is irregular, with elevations and depressions, or not having any great and uniform inclination, the crib can be sunk until it almost reaches the higher point or points, and while suspended in this position broken stone can be dropped into the pockets and around the outside, the stone assuming its own slope below the crib, and this continued until the crib is found to be uniformly and solidly supported; the masonry can then be commenced. Cement can be forced through pipes between the broken stone, thereby forming a solid and compact mass. (See Elevations, Fig. 23, (*a*), (*b*), and (*c*).)

COFFER-DAMS.

6. If the material composing the bed of the river is gravel, sand, clay, or silt, and either too soft to build upon, or is at all likely to be scoured out, instead of the preceding methods, the space to be occupied by the structure must be surrounded by a water-tight dam of some kind, so that the water can be pumped out of the enclosed space, and the excavation and preparation of the foundation-bed proceeded with as on dry land. The structure for this purpose, of whatever material it may be constructed, is called a coffer-dam. If there is no material depth of water, not exceeding 5 ft., and no current, clay either alone or mixed with sand and gravel can be dumped in the water, so as to form an earthen dam entirely around the space to be enclosed, and carried up 2 or 3 feet above the water surface, finished at least 3 ft. wide on top, the earth assuming its own slope below the water surface; this slope will be rather flat, from two to three horizontal to one vertical. The material to be excavated,

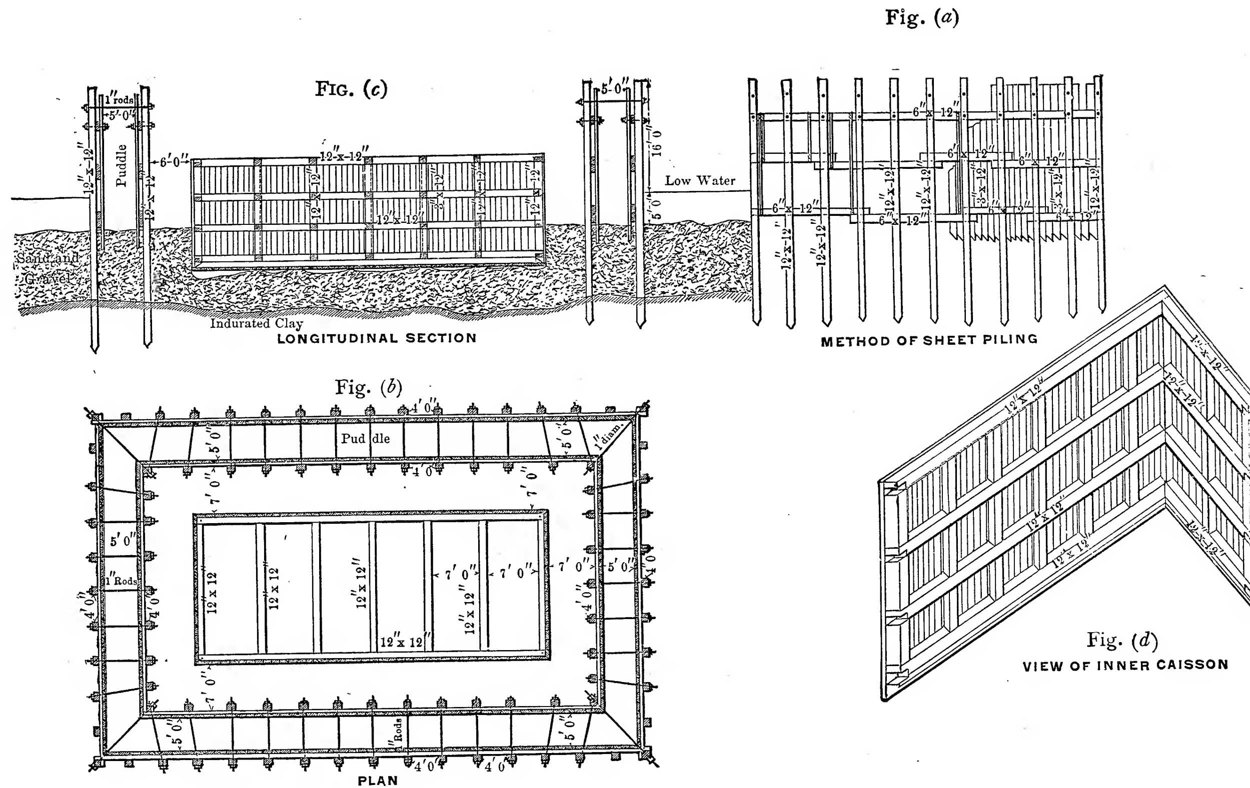


FIG. 21.—COFFER-DAM WITH INNER OPEN CAISSON OR CRIB, OHIO RIVER BRIDGE, POINT PLEASANT, W. VA.

[*To face page* 151.]

being saturated with water would, also require a long flat slope, consequently the area enclosed should be large in comparison with the area of the base of the structure; for instance, if the base of the structure is to be 20 ft. wide and the depth excavated is 15 ft. below the water surface, the interior width of the dam should not be less than from 80 to 110 ft., and the length generally from two to three times the width. Owing to the great dimensions required, this kind of dam is rarely used, and resort is had to the ordinary timber-coffer dam, constructed as follows:

ARTICLE XXX.

COFFER-DAMS OF TIMBER.

7. TWO rows of guide-piles, the piles of the proper length and at least 12 inches in diameter at the larger end, are driven entirely around the space to be enclosed. The area of this space should be considerably greater than the largest area of the structure to be built. If the dimensions of the base of the structure are 20×43 feet, the inside dimensions of the dam should under no circumstances be less than 6 feet more than the above, or 26×49 feet, and in general should be governed by the depth to be excavated below the bed of the river, the increase being at least equal to the depth, and better if equal to $1\frac{1}{2}$ times the depth; or for a depth of 10 feet below the bed of the river the dimension in the above case should not be less than 30×53 feet, and economy will justify an increase to 35×58 feet. More failures in coffer-dams result from the fact that the enclosed area is made too small, than from any other cause. A false idea of economy in the beginning generally results in much loss of time and a largely increased expenditure in the end. The dimension of the dams having been settled, the two rows of piles are driven so that the piles in each row will be from 4 to 8 feet apart, and the rows to be from 5 to 8 feet apart, according to the height of the dam above the bed of the river. This clear distance between rows

will allow from $3\frac{1}{2}$ to $6\frac{1}{2}$ feet in thickness of the clay puddle. This width is required to give stability to the dam. From 18 inches to 2 feet of good clay puddle is ample to prevent leaks. Wale-pieces, that is, horizontal pieces of timber, generally 6×12 inches, and of varying lengths, are bolted to the rows of piles, facing each other between the rows. Bolts, 1 inch diameter and from 7 to 9 feet long, tie the rows together. These are placed above the water surface. Another set of wales should now be placed, resting against the piles at or near the bed of the river. This is done by fastening battens to a wale-piece of any length, and forcing the wale-piece to the bed of the river, the battens then spiked at their upper ends to the top wale-pieces. This is carried all around both rows of piles, leaving spaces or gaps between the ends of the wales. Then other pieces are lowered in the same way, resting on top of the first pieces, and covering the vacant spaces between them. Intermediate rows of wales should be placed as above described, so that the vertical distance between any two sets of wales should not exceed 6 feet. Sheet-piles (planks of about $2\frac{1}{2}$ to 3 inches thick, and of lengths depending upon the depth of the water) are now driven, either by a heavy mall or a light hammer guided by leads as in pile-drivers, close together and resting against the wales. These should penetrate from 18 inches to 5 or 6 feet into the bed of the river, depending upon the material in the bed of the river. These sheet-piles are sharpened at the lower end, so that the bevel extends the entire width of the plank, i.e. from 7 to 12 inches, and when driven this bevel tends to hold each plank up against the last one driven. This forms a double close sheeting entirely around the enclosed space. Each plank when driven is spiked to the upper wale-piece. This then leaves a space to be filled with the puddle, varying from $3\frac{1}{2}$ to $6\frac{1}{2}$ feet in width. The guide-piles should be driven well into the bed of the river, and if practicable should pass through any sand or gravel into clay; but if the dam is made large enough, a penetration of from 10 to 15 feet into the bed, of whatever material that may be found, will in general be sufficient. The puddle can now be thrown in between

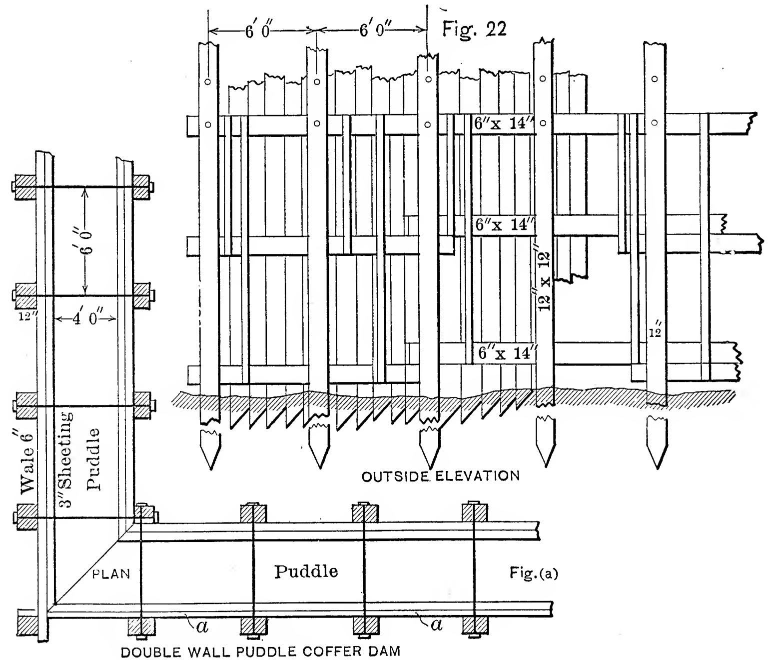


FIG. 22.

[*To face page* 152.]

the sheeting, and should be rammed or rather cut with a rammer head of 3-inch plank trimmed to a wedge-shaped edge. This prevents the formation of distinct layers. Each is cut into the layer below, binding the entire mass, and has a similar effect to the ribbed roller used in making reservoir embankments. The dam is now ready to be pumped out. Many authorities say that the soft and loose material between the sheeting should be dredged out. The writer does not comprehend the meaning of this. It is not necessary if the bed of the river is clay, nor is it necessary in gravel and sand, this being considered by many as the best material with which to puddle. If alluvial soil or silt, this is good puddle itself, and is not only water-tight, but often air-tight. It can hardly be necessary to do any dredging, unless limbs of trees or brush should be encountered. These would conduct water through the dam, and might cause dangerous leaks. The writer at least never did any dredging for this purpose, and has had good success in the many dams constructed by him. The construction of other forms of dams will be described before entering into a description of pumping and excavating, as these process will be the same for all. (Figs. 21, (*a*), (*b*), and (*c*); 22, and 22 (*a*).)

8. Sometimes four rows of wales are used, these being placed both on the outside and inside of the rows of piles, and the sheet-piles are driven between the wale-pieces. This guides the sheet-piles to some extent while being driven, but has no other advantage; requires more timber, is good practice, though not necessary. Sometimes the sheeting, instead of being 3 inches thick, is as much as 8 or 10 inches thick, with a tongue cut on one face about 2 to $2\frac{1}{2}$ inches broad and about the same depth, and a similar groove cut on the opposite face, and then driven so that the tongue of one piece fits into the groove of the adjacent piece. This certainly causes much expense in framing, and also delay in driving, and great waste of timber. This can never be necessary if a double wall is used, but will make a good single-wall dam, but will require strong bracing on the inside. There is probably no economy in this plan. Sometimes a groove is cut on both faces. The sheet-piles are then driven as close together as possible, and a 2-inch plank is driven in the grooves. This will close any opening that may exist by the piles leaning from each other in driving, and is to be preferred to the first or tongue-and-grooved method; or strips are spiked to the face of the piles, one strip on one face forming a tongue, and two strips on the other forming a groove, and driven as described. This prevents waste of timber, but is not as good as either of the other plans, unless the timber is very soft or splits easily, in which case the strips spiked on will be stronger than the regular tongue and groove. The writer thinks that the ordinary puddle-dam will prove more economical, more expeditious, and more satisfactory than either of the three last-mentioned methods. (See Fig. 23 (*B*), (*C*) and (*D*).

9. A solid wall of timber, either made of 12 × 12 in. sticks or plank, laid horizontally on top of each other and spiked or bolted together, which can be framed floating on the water, and large enough to enclose the required area, can be built and sunk into a dredged hole, or resting on the natural bed of the river, and sheet-piles driven all around and close against the timber wall to which they are spiked or bolted, will make a good dam, but will have to be strongly braced on the interior. This dam, when built in a circular or octagonal form, as is required in case of pivot piers for drawbridges, has many advantages, is easily and rapidly constructed, contains a minimum quantity of timber, will require little or no bracing on the interior, and, even if requiring large guide-piles to be driven on the inside to stiffen and hold it steady, will prove economical. This will make a good dam, when the bed of the river is a rocky ledge, by sinking it on the rock and throwing clay puddle all around on the outside, unless the current is so rapid as to wash away the puddle. In this case a double wall built as above described, and connected by cross-pieces of timbers for strut and tie braces, dovetailed or bolted to the walls, and the space between the walls filled with puddle, will make a stronger dam than any other described; can be rectangular, octagonal, or circular in plan; of any size or height required; will

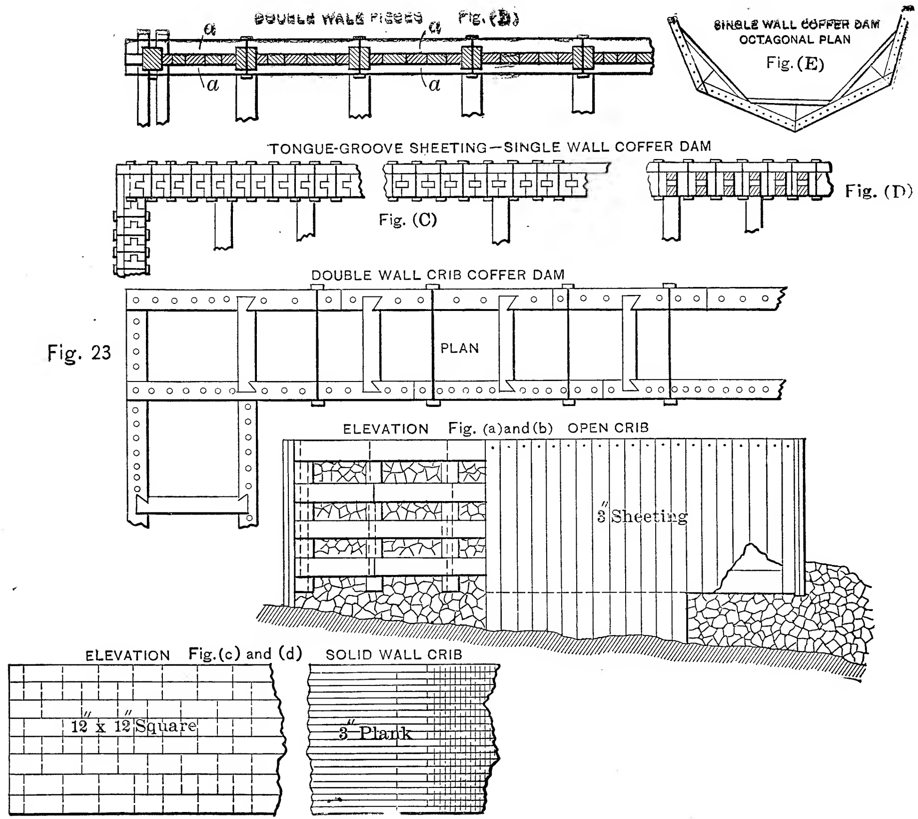


FIG. 23.

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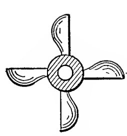
need little or no interior bracing. Sheet-piles should be driven in earth bottoms as deep as practicable, and on rock should be driven so as to broom or batter the lower ends, so that they may conform to the irregularities of the bottom; this will hold the puddle, and to a large extent prevent leaks along the rock under the puddle. Where the rock has a regular inclination or slope this crib-dam can be easily built so as to conform to the slope of the bottom. It should always be used in case of a rocky bottom. It is called the crib coffer-dam. In very rapid currents it can be built in sections of short lengths shaped as truncated wedges, alternate sections held and sunk in place and heavily weighted, the closing sections then floated and forced into their places by the force of the current, and then arrangements for holding the puddle can be made by uprights and sheeting in the enclosed space. This method can be used where the above-described method scould not be used. (See Fig. 23 (*E*.)

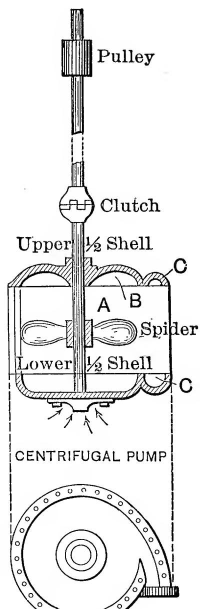
10. Mr. J. E. Robinson has a patent dam which has some merits worthy of notice, which does not, however, seem capable of economical application, except in shallow water and where no great current exists. It is to be always circular in form, regardless of the shape of the pier. It is constructed as follows: A series of shears or three pieces of timber held together by bolts passing loosely through the pieces at the top, allowing the legs to be spread out at any angle with each other; these are set up at intervals on the circumference of a circle; each has a small block and tackle fastened to its apex; large sheets of iron plate are then suspended under the shears. These plates are bolted together, forming a circular sheeting; inside of this long timbers, 12 in. × 12 in., slightly bevelled, as in arch stones, are driven with a light hammer, close together and resting against the iron sheeting. The space thus enclosed is ready then to be pumped out. This certainly forms a strong dam, even without interior bracing, when properly constructed, but in considerable depths radial bracing will be found necessary to prevent the bottom of the timbers from pressing inwards. The objections or defects arise mainly from driving

the piles, as it is difficult to keep them in contact for their full length, and they will almost always separate or spread at their lower ends; this can, at least in part, be overcome by cutting grooves in the piles and driving filling pieces of iron or wood in the grooves, as previously described. Owing to the form of the dam, the excavation is confined to a narrow area in the direction of the length of the pier, coming close to the sides of the dam along one diameter, and leaving broad, unexcavated spaces on either side. In addition, the excavated material is for convenience thrown on top of the undisturbed earth; the tendency of this is to bulge the sides out, and the great excess of outside pressure, in a direction at right angles to this, tends to force this part of the dam in, and aids in forcing the other parts outwards This distorts the entire dam; a portion of the dam comes in and another portion goes out, causing leaks that are hard to be controlled. In the writer's experience these dams gave great trouble and caused much delay; but in one case, at the Schuylkill River, this was used where an ordinary coffer-dam had failed entirely, and under circumstances peculiarly trying to a coffer-dam. This will be alluded to in another paragraph.

11. In whatever manner the coffer-dam may be constructed, with or without the puddle, the first step is to pump out the water This can be done by any of the ordinary pumps, such as the force, lift, or centrifugal pump. A single pump discharging a stream of from 4 to 10 inches in diameter should always be sufficient to keep the water out of a coffer-dam, and will ordinarily prove sufficient; but if the dam is badly constructed, or unexpected and large leaks are developed, it may require several pumps. There are two forms of the centrifugal pump: one in which the vanes are in a horizontal casing which is placed at the bottom of the excavation, and is lowered as the excavation proceeds, the discharge connected with this, and lengthened as required; this forces the water upwards and discharges it over the top of the dam; in the other the vanes are in a vertical casing, and is generally placed on or near the top of the dam; a pipe extends to the bottom of the excavation and



PLAN OF SPIDER
Fig. 2



PLAN OF SHELL

Fig. 3A

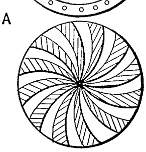


Fig. 24

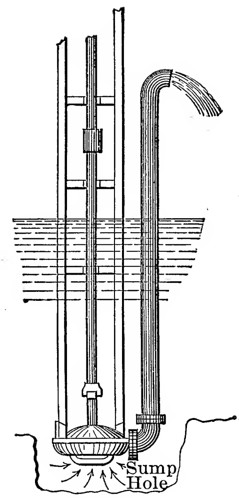


Fig. 25

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is lengthened as required; this lifts the water to the top and discharges it over the dam. Either of these pumps will throw a 6 or 10 inch stream, and should be ample for any dam properly constructed. The force-pump is placed either on top of the dam, or at the bottom of the excavation, or at any intermediate point, and will throw any diameter of stream required. The force-pump is apt to give more or less trouble by the accumulation of small fibres of wood, leaves, or small gravel and sand in the valves and between the sliding plates, causing delay and frequent stopping of the pump, often at a critical period of the work, no matter how carefully the suction end of the pipe may be protected by screens and strainers. In all important works duplicate pumps should be on hand, unless they can be obtained without delay, as the stoppage of a work of this kind might cause a suspension of the work for a season, or the breaking and loss of a coffer-dam. A centrifugal pump is less liable to get out of order, as it will readily discharge small chips, sand, and grit without damage or stoppage to the working of the pump. (Figs. 24 and 25.)

12. When the water is pumped out, and no serious leaks have developed, the excavation of the bottom can be commenced. The material, as far as practicable, should be kept piled up against the sides of the dam, and if the proper area has been enclosed there will be little danger of undermining the dam, or of forcing in the sides of the dam by the outside pressure, when the distance from the water-level to the bottom of the excavation does not exceed 25 feet, and this without interior bracing, in a double-wall puddle-dam. Braces should always be omitted, if possible (but in all cases timber should be kept in convenient positions, and of proper lengths, so that they can be readily and rapidly used if any signs of yielding are observed), as they materially interfere with and delay the construction of the masonry. A few braces placed diagonally across the corners of a rectangular-shaped dam will add greatly to the strength of the dam, and will be practically no obstruction to the work. It will be found advisable, both to limit the amount of excavation, increase the space on which the excavated material can be deposited on the inside of dam, and at the same time increase the height to which it may be piled against the side of the dam, to place a row of timber around the area to be occupied by the masonry, leaving a small margin all around, and then to drive sheeting behind this. As the excavation proceeds drive the sheeting farther down. In a large coffer-dam in the Ohio River the writer placed a double row of 12 × 12 inch timbers around the area required to be excavated, as soon as the water was pumped out. On the outside of this a 3-inch plank was bolted so as to leave a space of about 4 inches between the timber and the plank. Sheeting plank from 6 to 8 feet long was then driven two or three feet into the bed of the river. This provided a space of about 6 feet all around upon which the excavated material could be deposited. As the excavation advanced a man was kept driving the sheeting down. The plank bolted to the square timber prevented the upper end of the sheeting from inclining outward, and consequently the lower ends from pressing inwards, and the excavation continued through about 8 feet of gravel and sand to the clay. By this simple arrangement the excavation was confined to the exact area required. The material was simply cast by the shovel against the sides of the dam, both of which reduced the cost greatly, and the dam was well supported from the inside. Leaks were almost entirely prevented, and not a brace was used from the beginning to the end. The depth from the water-level to the bottom of the excavation was about 23 feet, the length of this coffer-dam on the inside 60 feet, and width 34 feet. The thickness of the sides of the dam was intended to be 8 feet, but, owing to careless driving of piles, it was not more than 6 feet in places. The thickness of the puddle itself varied from 2½ to 5 feet.

13. In another dam somewhat carelessly constructed, and which showed evident signs of weakness, of about the same size as the above, but requiring a wider and longer pier, an inner crib was constructed, while the dam was filled with water as follows: Horizontal timbers were framed together so as to enclose a space somewhat larger than the bottom area of the masonry. The bottom layer was cut diagonally, so as to form a cutting edge. Another 12 × 12 inch layer was bolted to this. 3-inch plank 8 feet long was then spiked on. At the corners and at intervals on the sides and ends, posts 12 × 12 inches × 4 feet were placed vertically on the horizontal pieces, to which the plank was spiked. On the posts another course of horizontal timbers was placed, and so on until the crib rested on the bottom. The space between the crib and coffer-dam was then filled with earth. The water was then pumped out. A few braces were placed on the inside of the crib; the excavation was then commenced, the crib, weighted with large stone, settled gradually. After excavating a few feet, the coffer-dam commenced to be undermined; the material between crib and coffer-dam commenced to flow to the interior. The sides of the dam bulged inwards until they rested against the crib. The puddle in the coffer-dam settled; the pumps could not keep down the water. It was decided to flood the dam for fear that the crib-dam and braces could not stand the pressure. After carefully considering the conditions, two plans presented themselves. One was to build a new dam; this would take a long time, cost a great deal of money. The other was to re-puddle the old dam, and also to throw a large quantity of puddle around the old dam on the outside, pump the water out, and then brace strongly the crib, and endeavor to reach a safe foundation, the inner crib holding the coffer-dam in place. This was the most expeditious, and seemed practicable, if we could pump the water. This was decided upon and acted upon at once. The water was again pumped out. Bracing the crib strongly as the water fell. The dam rested hard against the crib in places, creating great frictional resistance, requiring a largely increased weight to sink the crib. The excavated material was placed between the crib and the dam; it would continually flow back. Such material as was not placed between the crib and dam was lifted out in buckets and dumped into the river. In this manner, with much delay, and with slow progress, we succeeded in reaching the clay upon which the structure was built, after excavating through 14 feet of gravel and sand. This inner crib has the following advantages: 1. It reduces the amount of material to be excavated to a minimum, saving time and money. 2. It enables us to keep the lower part of the main and sheet piles always covered, preventing undermining of the dam, and great inflow of water, sand, and gravel, by keeping the space between the dam and crib always filled with material. 3. The crib can be sunk even below the points of the main or guide piles without danger. The space between the crib and dam should not be less than 5 or 6 feet. Great depths can be reached by this means, and the crib should always be used if the depth is over 20 to 25 feet. The crib can be built up as the excavation and sinking progresses. (See Fig. 21, (*a*), (*b*), and (*c*).) and (*d*).)

14. In many cases a crib of this kind can be used without any puddle-dam on the outside. This would require the plank sides to be calked to prevent too great an inflow of water. A modification of this construction is used as a coffer-dam, for the sides of the open caisson, or on top of a pneumatic caisson, and of heights from 20 to 40 ft. This will hereafter be explained and illustrated.

15. The puddle filling in coffer-dams is generally composed of such earth as is easily accessible. Some materials are better than others, but any ordinary clay or loam will make good puddle if the layers are well bonded or cut into each other. What is known as brick-clay makes an excellent puddle. Some high authorities recommend a sand and gravel filling, which is claimed to be better than clay; and should a large leak occur, the sand and gravel will fall and fill the cavity, and can be refilled on top. This is true; but water will always find its way through sand and gravel, where, if well mixed with clay, or clay alone, it is practically impervious to water. The writer prefers greatly this last material.

16. Coffer-dams generally prove to be expensive, are always uncertain, and, unless sufficiently large, or some form of inner support used as above described, will give trouble, frequently

filling up with water and earthy material, or undermining, and not unfrequently breaking in.

17. Sometimes coffer-dams are used when it is intended to drive piles in the enclosed space for the foundations, no material being removed from the bed of the river, but simply to keep the water out while the piles are being cut off and a timber platform framed on top. This platform consists of several courses of square timber. The piles being cut off at the same level, they are then capped by 12 $\times$ 12 in. timber, then another course at shorter intervals placed across and at right angles to the caps, and over this another course of square timber placed close together, forming a solid timber floor, the whole thoroughly bolted together with drift-bolts. The open spaces are sometimes filled with broken stone or concrete, around the tops of the piles, between and under the timbers; this is not necessary unless the material is very soft or yielding, and is only intended in this case to give lateral stiffness and steadiness to the piles; or the timber platform may be omitted, and a thick bed of concrete placed around and over the top of the piles.

18. If the material has been removed to the proper depth, the bed of gravel, sand, or clay is levelled. The masonry may be commenced directly on these materials, but it is better to first lay a bed of concrete not less than 2 to 3 feet in thickness over the area, and extending a distance equal to the thickness, outside of the space to be occupied by the masonry; or a course of square timber may be laid on the foundation-bed; or the two may be combined, timber being embedded in the concrete.

19. Coffer-dams are rarely used where the material has to be excavated to any great depth, for the reasons above described. Some other method will be in general preferred, such as the open caisson.

ARTICLE XXXI.

OPEN CAISSON.

20. AN open caisson is simply a water-tight box open at the upper end, and is constructed as follows: A floor of square timber, 12 × 12 in., built of two or more courses of timber, well bolted together, and of any size and shape, but from 4 to 6 ft. larger than the proposed structure; this floor is to be thoroughly calked. Near the outer edges of this, large bolts with eyes are fastened; about one foot inside of the bolts square timber sills 12 × 12 in. are placed; vertical posts 12 × 12 in. are connected to these by mortise-and-tenon joints. On the top of these posts, which are placed from 4 to 5 ft. apart, caps of 12 × 12 in. timber are placed and secured; these skeleton frames are then covered by two courses of sheeting plank, the inner course placed diagonally, the outer course horizontally; over the top of the sides 12 × 12 in. pieces are placed, projecting over the sides and ends not over 1 to 1½ ft. Long iron rods with hook at one end and screw-threads at the other are hooked to the eye-bolts and passed through holes in the top pieces; a washer is placed on top, the nut being screwed on so as to bring the sides to a hard bearing on the floor or platform. These sides can be of any height, but it is best to use a height of not more than 15 to 20 ft. at first, and when sunk to this depth add another section similarly constructed on top. In this case a thimble or sleeve with right- and left-handed threads should be used instead of the nut on the end of the rod, so that other sections of rods can be connected as the sections of the sides are added. As soon as the sides are brought to a firm bearing on the floor the outside planking should be thoroughly calked.

21. The caisson is now ready to be floated to the site of the structure. A course or two of masonry should be laid to steady the caisson and prevent any tendency to careen or turn

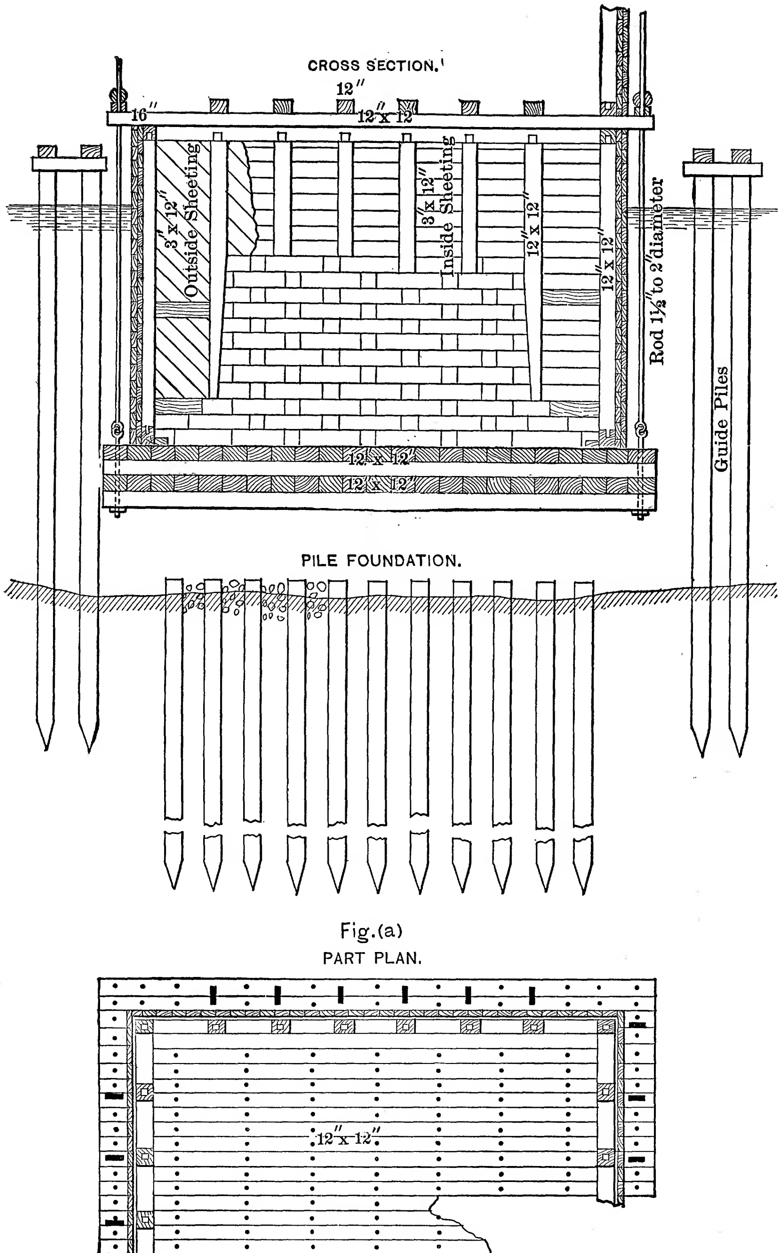

Fig. 26.—Open Caisson.

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sideways. If the bed of the river is of a firm material—clay, sand, or gravel—it is only necessary to level the bed. If there is danger of scouring when the current is strong, or the material is too soft to bear the load, piles must be driven and then cut off to a level either at the bed of the river or at some point below the water surface. In either case the building of the masonry is continued until the caisson nearly reaches the bed of the river or the top of the piles, as the case may be. The caisson carefully adjusted or located in its exact position by ropes attached to anchors or to guide-piles driven for the purpose, enough water is now let in to complete the sinking. If it does not rest in a level and easy position, or veers out of place, the water is pumped out, the caisson lifts, the bed or piles properly levelled, and the caisson again sunk; and this must be repeated until the result is satisfactory. In the case of resting on piles, a diver should be sent down to see that it rests practically on all the piles, as with a light load it might be supported by only a few piles, and the increased weight would cause the piles to sink; but even in this event the only effect would be to settle until other piles were brought into a bearing, and any increase of height of structure can be supplied by the courses of masonry. Any number of piles can be sawed off under water so as not to vary in level more than one-quarter inch; this difference can cause no harm. The structure can then be completed. When this rises above the water surface the iron rods can be unhooked and removed; the sides of the dams can then be lifted and used on other piers. To enable this to be done the corner posts are made in two pieces and held together by a pin-bolt, which can be removed, and the sides and ends easily fall apart. The sides of the dams should be braced against the masonry by short blocks as the caisson sinks. It will hardly prove economical to use the sides over again, unless timber is scarce or costly. The manner of cutting piles off under water will be explained under head of Piles. (Figs. 26 and 26 (*a*).)

22. In depths of water not exceeding 20 to 25 feet this method of construction is simple and economical. The bottom of the caisson forms a part of the permanent structure. The masonry is being built up as the caisson sinks, which is not the case in a puddle coffer-dam. The material can be cheaply excavated and levelled by dredging, providing that a dredging machine can be hired at a reasonable cost, or enough of it is to be done to justify a contractor to undertake the work. These sides are properly a single-wall coffer-dam, calking being substituted for the puddle, and interior bracing used instead of thickness of dam to secure strength and stability.

ARTICLE XXXII.

CUSHING CYLINDER PIERS.

23. THESE piers are constructed as follows: A cluster of piles is driven, to a solid bearing or to a satisfactory resistance, as close together as practicable, and in juxtaposition,* if it can be done; each cluster is composed of from 4 to 12 piles, bolted together strongly at or near their upper ends. The piles should be straight and of good size, not less than 12 inches square or diameter at large, or less than 9 or 10 inches at smaller end. In driving such clusters the pile should be slightly inclined or raking while being driven, and the tops subsequently pulled together and held by bolts. The piles should penetrate the soil not less than 20 feet, and as much more depending on the nature of the material as to softness or hardness, and should reach well below any danger from scouring action. Iron cylinders from 4 to 10 feet in diameter should then be sunk around the piles at least to a depth of 10 feet below the bed of the river. These cylinders are made in sections of from 5 to 10 feet in length and of a thickness from 1 to $1\frac{1}{2}$ inches of metal, with internal flanges about 3 inches wide. The lower edge of the bottom section should be brought to a feather or cutting edge. Cast-iron is generally used. Enough

* The same number of piles, if driven at small intervals apart, would carry greater loads, as a larger aggregate area of surface would be available for frictional resistance, but require larger cylinders.

of these sections are bolted together to reach from the bed of the river to a point above the water surface. This being placed around the pile clusters, another section is bolted on. The material is then removed from the inside or stirred and loosened so that the cylinder will either sink by its own weight or by adding weights on top. When it reaches the proper depth, the interior of the cylinder is filled with concrete. The water may be pumped out if the material at the bottom is clay or silt, and in some cases if of compact sand; but commonly the water remains and the concrete is simply thrown in. When filled a thick iron cap from $2\frac{1}{2}$ to 3 inches thick is bolted to the top. For that portion of the cylinder above the bed of the river the flanges can be cast on the outside. Two of such cylinders and pile clusters, bolted and braced to each other at the proper distance apart, constitute a pier. Four may be used to a pier, which greatly increases the stability. A two-cylinder pier, if it stands any great height above the bed of the river, is wanting in lateral stability; and if the obstruction caused by the piers causes any scouring, the safety of the structure is greatly endangered, calls for frequent examination, and requires for security the constant and liberal use of riprap or broken stone around the cylinders. The piles should be cut off at or below low water. In this form of pier the piles are the real supporting power. The concrete carries the load from the structure above to the piles; the cylinders merely serve as a casing to hold the concrete. If in sand or gravel, the friction on the sides of the cylinder will give some additional support. These piers are economical, easily and rapidly constructed, but are wanting in stability. (Figs. 27 and 27, (*a*).)

24. In many cases, both for railroad and highway bridges, the piles are omitted; the cylinders are sunk into the material anywhere from 3 to over 100 feet, and are then filled with concrete. In such cases the cylinders are of large size for railroad bridges, are generally larger at the bottom, and taper or batter to the top. They are capable of bearing heavy vertical loads, are wanting in stability, especially if subjected to heavy lateral pressures resulting from moving ice and drift in large masses, or exposed to great and repeated shocks. Their cheapness has led to a frequent use for highway bridges and comparatively short spans. Some examples will be given in another paragraph of such piers, with methods of sinking, dimensions, heights, etc.

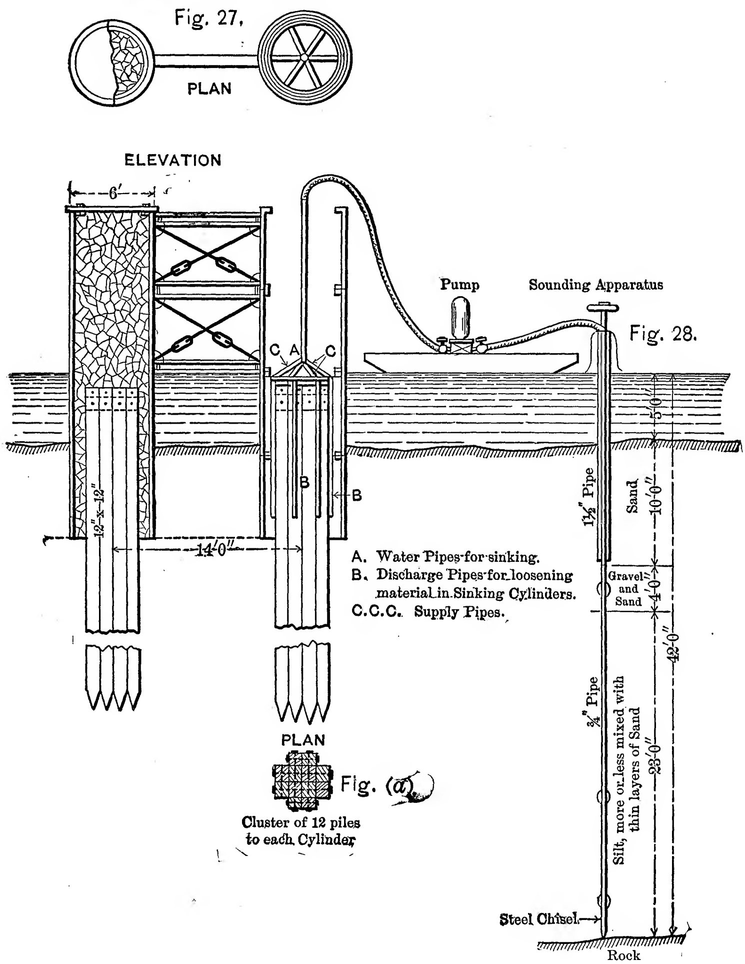
ARTICLE XXXIII.

SOUNDINGS OR BORINGS.

25. THE importance of a thorough knowledge of the material underlying the site of any proposed bridge is so evident and great that it would seem unnecessary to more than allude to the subject; but the fact is that engineers, from a notion of false economy, neglect to gain the necessary information, and as a result design and prosecute the construction of even important structures either ignorant or at least with but very meagre knowledge of the nature and lay of the strata, and when too late find themselves involved in difficulties which will cost thousands of dollars to overcome, which could be saved by the expenditure of not over two to three hundred dollars, expended in judicious and thorough soundings, to say nothing of the delay and stoppage of the work. Thorough sounding will always be money well spent and time saved. For these reasons this subject will be explained in some detail.

25½. The first and usual method is simply to drive an iron rod or pipe from 1 to 1½ inches diameter. A rod of this kind can sometimes be driven to a depth from 10 to 30 feet by constant hammering and turning after each blow. The information, however, obtained is meagre and unsatisfactory; the character, thickness, or lay of the different strata penetrated cannot be determined; boulders, even small ones, will stop the driving, as will also logs or drift or wrecks. Reports based upon such soundings lead to erroneous conclusions; designs, estimates, and contracts are made that require change and alterations, involving often largely increased cost and delays. In one important bridge across the Ohio River, plans,

CUSHING CYLINDER PIERS



estimates, and contracts were closed on the supposition that rock would be reached at a short depth, and this information was apparently confirmed by the fact that another bridge in sight was built on rock at a short distance below the water surface; but subsequently it was discovered that no firm material existed under from 60 to 70 feet below low water, and the foundations had to be constructed by the pneumatic process. Every engineer knows the cost of such a change in the conditions and terms of a contract.

26. A better and more satisfactory method is to sink a terra-cotta or iron pipe from 3 to 8 in. in diameter, as follows: The pipe is pressed into the surface as far as practicable; then a long narrow bucket with a cutting edge, and a flap valve a little distance above the cutting edge opening upwards, is dropped into the pipe and is alternately and rapidly raised and dropped. The material will be collected in the bucket, and at intervals the bucket is lifted entirely out and the contents of the bucket are emptied. This is repeated; the pipe will gradually sink, a man standing on top if it is necessary. Other sections of pipes are added from time to time. It will be necessary sometimes to pour water into the pipe to aid in the cutting and flow of the material into the bucket. The bucket should be connected by a rope passing over a sheave connected with a frame or shears above. Great depths can be reached by this method with reasonable rapidity, and at no great cost. The writer used this on the Ohio River with satisfactory results. It enables you to determine the thickness and nature of the strata; that is, you can determine whether the strata is sand or gravel or clay, but you do not know in what condition it exists. Clay will be brought up in the condition of mud. The strata may contain a large quantity of water, which would render it unsuitable for a foundation-bed when the overlying strata are removed. An experienced well-borer can form a good opinion on these matters. On the whole much valuable and satisfactory information can be gained, and it is greatly to be preferred to the rod process. The rod will often bend above or below the ground, and this often misleads, as the bending apparently renders the depth actually penetrated uncertain.

27. The third method is simple, relatively inexpensive, and entirely satisfactory. All that is needed is a number of sections of pipe, $1\frac{1}{2}$ in. diameter and $\frac{3}{4}$ in. diameter, a small hand force-pump, and a small barge or raft. Threads are cut on both ends of every section of pipe, and a number of thimbles with internal screw-threads to fit. A chisel-steel wedge-shaped section about 1 ft. long, with two small holes passing through the faces of the wedge into a hollow tube, made to fit the $\frac{3}{4}$-in. pipe. An upper section also about a foot long has a solid handle at right angles; a short hollow elbow is connected with this, to which a hose from the pump can be connected. Enough sections of the $1\frac{1}{2}$-in. pipe are connected together to reach from the surface of the water to the bed of the river. This pipe is lowered to the bottom and pressed 4 or 5 ft. in the bottom; the top is then secured to the boat or raft. When the water is deep and the current is rapid, this pipe is liable to bend considerably, and might part at some joint. A rope attached to a small anchor and to the pipe at some point below the water will relieve this, the anchor being dropped well up stream. The $\frac{3}{4}$-in. pipe is now connected together with the chisel end at the bottom and lowered in the $1\frac{1}{2}$-in. pipe; the hose then connects the smaller pipe with the pump. The pump is started; the water rushes through the small apertures at the bottom, and passes upward between the two pipes, bringing the material of the bottom with it, which is discharged at the top. The small pipe is turned, and easily works its way into the soil. The larger pipe would also settle with it; but this is not necessary, and is fastened at the top. At intervals of 4 or 5 ft. the inner pipe can be lifted entirely out, the chisel end removed; a tube of iron or brass a foot long, slightly contracted at the lower end, of the size and shape of the straight cylindrical lamp-chimney, is now screwed to the small pipe, lowered to the bottom of the hole, and pressed 1 foot into the bottom, then lifted out again. The material in the brass tube can now be pressed by means of a round stick into a lamp-

chimney, a piece of thin sheet rubber fastened over the ends. We have now a cylindrical specimen of the material at that depth in the exact condition in which it existed in the strata; this will retain its moisture for a long time. This is, however, only practicable in clay, silt, or mixed soils. In these materials the hole will remain, and the pipe can be lowered to its bottom without letting the larger pipe follow. We have often bored 40 to 50 ft. in a day. When in a bed of loose sand and gravel, the water will not return upwards through the annular space between the pipes, but will escape laterally at the bottom, so that the material cannot be brought to the surface, but the water will so loosen it that the pipe will readily sink. It should, however, be turned constantly and rapidly, and be lifted a short distance occasionally; otherwise the sand and gravel will rise up between the two pipes, or compact above the chisel section and bind the pipe, increasing both the difficulty of sinking or raising the pipe, and causing sometimes the loss of the smaller pipe. It is better in this material to let or make the larger pipe follow the smaller one, but it is not necessary. The only skill required in this method of boring is to prevent this binding. In silt there is danger of the pipe sinking too rapidly, which would also bind the pipe; constant turning and frequent lifting will prevent it in either case. If bowlders are encountered, or logs either, they can be readily drilled through, as the pipe is free to be lifted and dropped. Rock is easily determined either by hammering on the top of the pipe or lifting it and letting it drop; the rebound or the sound either enables you to distinguish between solid rock and bowlders. The entire outfit is cheap; any plumber or mechanic can make the connections necessary with the ordinary gas or water pipes in common use. Two or three men can perform all of the work required. This method was used in all of the piers at the Susquehanna River, Schuylkill, and Tombigbee, and Ohio River at Louisville, in depths varying from 40 to over 100 ft. A single boring at the site of a pier is not enough; at the Susquehanna and Schuylkill, from six to ten soundings were made at the site of each caisson, one at each corner, and one or two intermediate along the sides, and if any great or abrupt irregularities were developed additional borings made. From this an exact diagram of the lay of the bottom, in reference to the surface of the water and bed of the river, was made. We knew in the beginning the exact nature of the material to be passed through, the high and low points of the rock and the exact depth to each, and the exact amount of material to be excavated. The position of each sounding was located accurately by triangulation or the wire measurement from an established base. At the Susquehanna a good part of the boring was done in the winter, when the river was solidly frozen over. The information thus obtained cost a very small sum, and was invaluable. (See Fig. 28.)

Article XXXIV.

TIMBER PIERS.

28. Where stone or brick is hard to obtain, or costly on account of the long distances over which it must be hauled or transported, or when it is important to erect a bridge without the delay incident to constructing masonry piers, it becomes necessary to build timber piers. These piers are generally so designed that masonry or iron piers can be easily and conveniently constructed at some future time. Two types of such piers, constructed by the writer, will be briefly described and illustrated. In building a railroad across the swamps in Alabama between Tensas and Mobile, a number of bayous or streams had to be crossed; these being navigable, many drawbridges had to be constructed, large enough to pass the large steamboats navigating the main rivers. The pivot or centre piers were constructed by driving a number of piles $2\frac{1}{2}$ ft. centres over a square area of the proper size; these piles were cut off a few feet above the water surface, were capped with 12×12 in. square timber and bolted to the piles with drift bolts 1 in. square and 2 ft. long. Upon these, and at right angles to them, other timbers were placed at about one foot intervals, and upon these a solid flooring of square timbers; these timbers were connected by 1-in. screw bolts with 2-ft. grip —that is, length of bolt between timber surfaces; the entire length from out to out of bolt, included head, nut, and washers, would be 2 ft. $3\frac{1}{2}$ in. This completed the pier; the upper surface was then planed to a level, to receive the turn-table arrangements. The number of piles required varied from 49 to 81, according to the size of the piers, and were driven from 30 to 40 ft. into the bed of the stream, which was composed almost entirely of a soft silt or mud. The rest-piers were composed of three rows of piles, about 3 ft. apart, from 5 to 6 piles in each row; these were capped with three courses of timber, as in the pivot pier; upon this flooring other timbers were placed to support the latch beam for the ends of the draw span, and for the support of the stringers of the trestle approach. These piers were now complete; they were constructed in a few days and at a small cost. It was an easy matter at some subsequent time to cut the piles off below low water, and to build brick piers on them; this has been done.

29. The following is another form of timber pier carrying spans from 100 to 120 ft. Sixteen piles were driven in two rows, eight piles in each row. The distance between the rows was regulated, as in masonry piers, by the size required at the top, and allowing for the batter. The distance between the piles in each row was regulated by placing the piles to the best advantage for supporting the structure above, as will be seen in the drawing; these were then capped with 12×12 in. timber, upon which were placed other timbers at right angles to the first, all bolted and fastened together by iron straps. Upon this platform a strong double framed-trestle was erected, constructed of eight vertical and inclined posts, with cap and sill; on these cross-pieces were placed, and then a platform for the bridge to rest upon. All timbers were 12×12 in. square, the whole structure well tied together, and braced longitudinally. See drawing, Fig. 30, (*a*), and 1*a*. Such a structure will bear a heavy load, is, however, temporary in its nature, is very light, and is wanting in stability if liable to be subjected

to severe shocks. In this case a timber starling or cut-water should be constructed at the up-stream end, either built entirely separate from the pier itself or forming a part of it. Extra piles can be driven triangular in plan, and resting upon these large square timbers should be placed, inclining at a rather flat angle to the body of the pier; this should be close-sheeted with plank, and filled in part or entirely with broken stone or gravel, or better with concrete—in which event, when the timbers rot, a permanent concrete pier would remain. Either pine or oak timber is well suited for such structures.

Article XXXV.

FRAMED TRESTLES.

30. Trestles are used to carry a railroad over swamps, rivers, or creeks, when material for making embankments is difficult to obtain. In building over low places, a simple framed trestle composed of a cap, sill, and four posts, two placed vertically under the rails and one on either side inclined to the verticals, with a batter of about 3 in. to each vertical foot. This frame ordinarily rests on mud sills, which are simply short pieces of square timber 5 or 6 ft. long, partly imbedded in the soil, or better on small rubble-masonry pedestals. All timbers in these frames or bents are generally 12 × 12 in. timber; the batter posts are sometimes 10 × 12 in. square. Diagonal bracings, called X-bracing, are then placed passing diagonally from the top to the bottom of the bent on both sides, made of 3-in. × 9 or 12-in. plank, and bolted to each post. Spikes are often used; bolts cost but little more, and are much better and ultimately more economical. If the bents are very high, longitudinal braces either of 3-in. plank or 6 × 6 in. square timber are placed from bent to bent and bolted to the posts. The bents are placed generally from 12½ ft. to 14 ft. centre to centre. Where a greater height than 25 ft. is required for the bents, they are built with one, two, or more stories, each of a

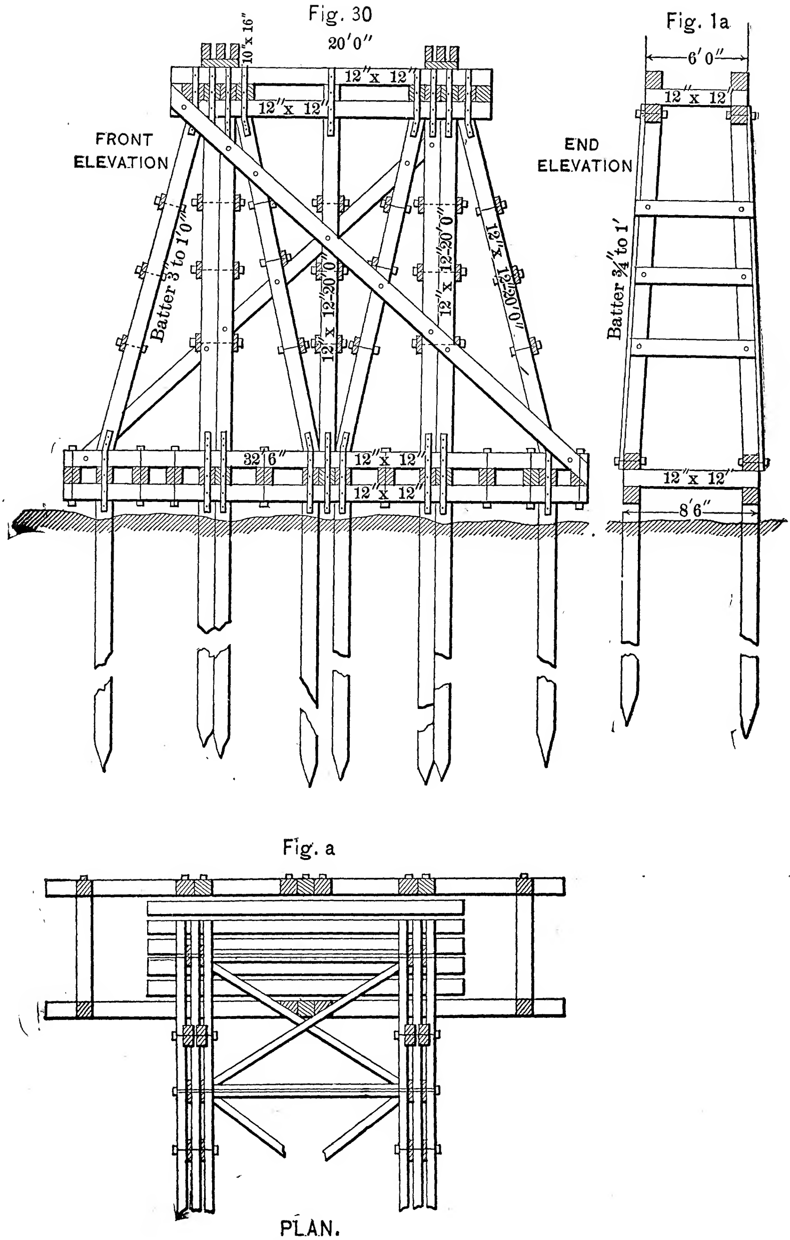


FIG. 30.—TIMBER PIERS.

[*To face page* 172.]

height not exceeding 20 ft., the upper section resting on top of the next lower, and so on to the bottom, the posts in each section framed so as to be in the prolongation of those in the sections above, and additional verticals or inclined pieces are introduced in the lower sections, the whole thoroughly bolted and braced together. In the form of trestle just described, the vertical posts are supposed to carry the greater part of the load, the batter or inclined posts mainly intended to give a wide base and lateral stiffness to the bent. This is the actual case, when the top of the batter post is a foot or more from the top of the vertical. When placed close together both posts bear a part of the load. In order to give stiffness and at the same time to make all posts bear an equal share of the load, all of the posts are inclined at or about the same angle; this is called the *M*-trestle; two posts touching at top and inclining downwards in opposite directions are placed under each rail, the inner posts coming together at the bottom in the middle; the outside posts may have a little greater batter than the inner ones. In this manner the load is distributed over and borne by four instead of two posts. In this form of trestle the posts need not be more than 9 or 10 × 12 ins. square; but unless the amount of timber required is very great, as in very high and long trestles, the timbers are not proportioned to the loads they have to bear, as the amount of material saved would be of little moment. In very high trestles, such as two, three, or more stories, economy requires the bents to be placed farther apart, the timbers in the bents remaining from 10 to 12 × 12 ins. square. In high trestles the bracing and some of the auxiliary or secondary members are of smaller scantling or plank. The drawings, showing in detail the types of the different trestles and the dimensions of the several members used, there will be no need of a lengthy description. (See Figs. 31 and 31(*a*).)

31. It will be sufficient to say that the upper cap need not be over 10 or 12 feet long. The intermediate sills (which are a sill to the section above and a cap to the section below) are equal to the length of the cap increased by 6 inches for each vertical foot between them, to these the posts are connected by the mortise-and-tenon joint, or by drift-bolts or iron straps, —commonly by the s2cond method. Some engineers use two ntermediate pieces, so that each section is a separate and independent frame, in which case the longitudinal braces are framed between the two. There seems to be no decided advantage in this, and it requires more material, and consequently more cost. The bottom sill will be equal to the top cap increased by 6 inches multiplied by the total height of the trestle. All the main posts should be in lengths of about 20 feet, except the bottom section, which is generally less than 20 feet, often not over from 5 to 10 feet. The longitudinal braces are from 6 × 6 inches to 6 × 8 inches square, and the same for the secondary members. The diagonal bracing is generally 3-inch plank, 9 to 12 inches wide. (See Fig. 32, (*a*) and (*b*).), and 34.

32. Whatever may be the height and form of trestle-bent, and whatever distance apart, there are placed on top of the cap, stringers, extending longitudinally and at right angles to the cap, one stringer under each rail, and commonly placed directly over a post. Each stringer is made of two pieces for bents $12\frac{1}{2}$ feet centres. They are 6 × 14 inches cross-section each, and should be 25 feet long. They are placed side by side with a 2 or 3 inch space between them, and bolted together by screw-bolts, blocks of wood or cast-iron spools being placed between them, through which the bolts pass. These bolts are from $\frac{5}{8}$ to $\frac{3}{4}$ inch diameter, with cast-iron or wrought plate washers, and nuts to suit. Four bolts are used over each cap, and sometimes two or more between the bents, for each packed stringer, and one bolt from $\frac{3}{4}$ to 1 inch diameter to bolt the stringer to the cap. This bolt has to be slightly inclined so as to pass the end of the post below. For bents 14 to 15 feet apart, packed stringers 7 × 15 inches or 8 × 15 inches are used, and if practicable to obtain them they should be 28 or 30 feet long; but if not practicable, lengths of 14 or 15 feet can be used. But this requires a bolster, that is, a piece of timber 8 to 10 inches thick, 4 to 5 feet long, placed on the cap and bolted to it. The stringers then rest on the

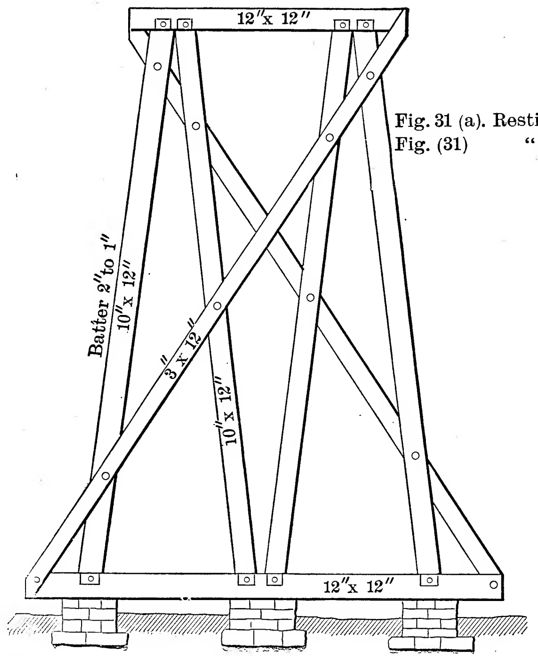


Fig. 31 (a). Resting on Mud-sills
Fig. (31) " " Rubble Pedestals.

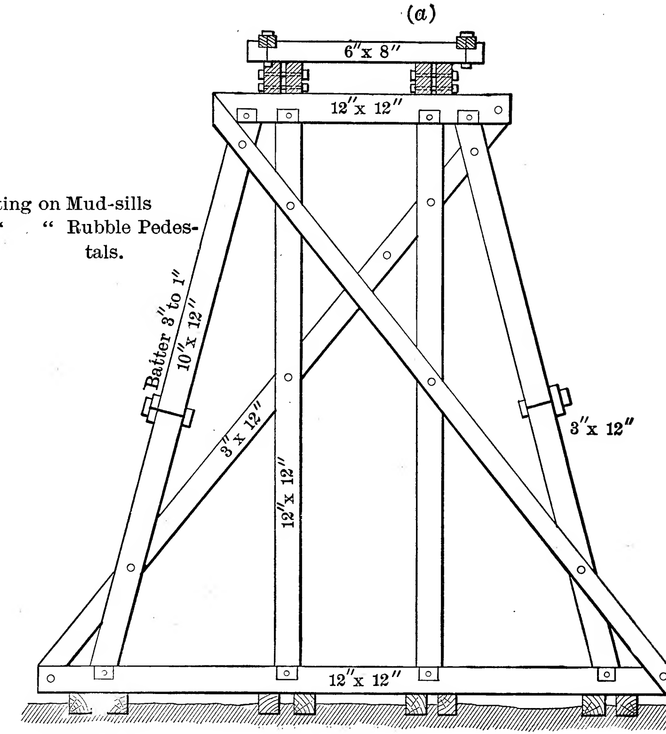


Fig. 31

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bolsters, to which they are bolted. In this case the stringers do not break joint on the caps. It does not form as stiff a trestle as when the joints are broken, but will answer every purpose. Pieces of timber bolted to the side of the stringers and resting on the cap can be used instead of the bolsters. (See Figs. 32 and 34.)

33. For spans from 15 to 25 feet it is economical to use stringers of the same cross-section as above, but they should be strengthened below either by struts resting on the bents at the lower end and at the upper end against straining-pieces—these are pieces of timber bolted to the stringers underneath and near the centre, these pieces being 5 or 6 feet long, and 6 or 8 inches thick—or by iron rods passing through the stringers at the ends, and under a block of wood or a short iron strut placed under and at the centre of the length of the stringer. Either of these constitute a trussed beam. The stringers in this case can be either of two or three pieces packed together. (See Fig. 32, (*b*) and (*c*).) Sometimes, instead of trussing the stringers as above, an increased number of pieces can be used. Four pieces of timber 12×12 inches square, bolted together with bolts and packing-blocks, which are framed into the sticks so as to act as keys, will be amply strong for a span from 20 to 25 feet; or two pieces laid side by side, and another single piece on top, will do for a span from 15 to 20 feet. These are mere expedients, have a bad distribution of material, and make a heavy, bungling-looking job. (See Fig. 33, (1*a*) to (6*a*).)

34. On top of the stringers the cross-ties are laid. These are 6×8 inches square, and from $8\frac{1}{2}$ to $9\frac{1}{2}$ feet long, are generally spaced from 12 to 16 inches centres, and spiked to the stringers. It is poor economy to space the ties with too great intervals. If a train runs off the track the wheels sink in the space between the ties, and before the momentum can be overcome, the trestle will be torn to pieces, and not unfrequently the train will be badly wrecked. If the ties are not over 2 or 3 inches apart, the train can easily run on the ties without serious damage to either train or trestle. On top of the ties and near their outer edges guard-rails (longitud-

inal pieces of timber 6 × 6 inches or 6 × 8 inches) are placed and bolted to the ties. A common rule is to use a ½-inch screw-bolt at every fifth tie, and spikes between the bolts. The guard-rails are sometimes cut or "dapped" to the depth of an inch, a tongue projecting down between the ties. Whatever theoretical advantage may exist practically as the work is done, there is no advantage whatever gained. The writer thinks that the guard-rail had better be placed inside and close to the rail, not over two or three inches from the rail. In this case the top of the guard-rail should not be above the iron rail of the track. It might be placed both inside and outside. This completes the trestle. The cross-ties on trestles and bridges are always sawed. On embankments they are hewn. It is claimed that a hewn tie will last longer than a sawed tie. No good reason seems to be assigned for this. The hewn tie is almost universally used on embankments, as they are generally cut and hauled from the woods adjacent to the line, carried by trains to point where used; smaller trees are used, which are younger and more vigorous than those which are larger and on the decline, and which have to be split through the centre. In the piney woods of the South dead-wood trees, either standing or fallen, are largely used for ties, and are said to be equally as good as, if not better than, green-wood ties, as the sap is either rotted or is easily removed. Oak ties, either white oak or post oak, are preferred to the pine ties. They are not so rapidly cut into by the rail, this cutting tending to cause rot in the tie under the rail, and they hold the spike better, owing to their hardness; but, on the contrary, they warp and split to a greater extent than pine, which admitting and holding water causes internal rot. Oak ties cost about 10 cents more than pine ties, 40 cents and 30 cents, respectively, being average costs. Pine is generally used where it is abundant; oak, where it is abundant. The same may be said in general of the other timbers of the trestle. But probably pine is preferred for caps, sills, and posts. It is more easily cut and framed, has great strength and durability, does not warp, split, or crack as readily as oak, and can be

Fig. (a)

Fig. (b)—Side Elevations.

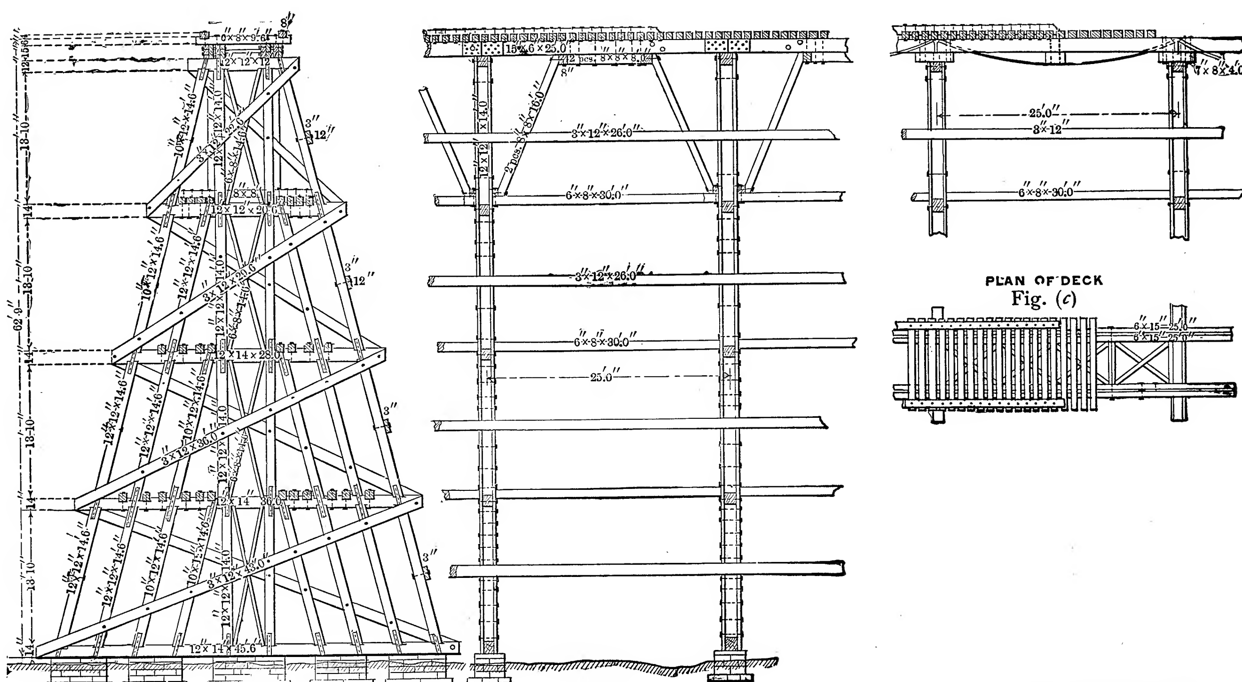


FIG. 32.

[To face page 176.]

obtained in longer and straighter sticks. Oak is somewhat stronger and heavier. Either will answer in any framed structure. Cost and rapidity of supply alone are factors worth considering in deciding between the two.

Article XXXVI.

QUALITIES OF TIMBER.

35. In the North white pine is abundant, and is used for all purposes in framing. It is soft and white, easily worked, and possesses great strength and durability. In the South the long-leaf yellow pine is found in abundance; being harder and generally claimed to be stronger than the white pine, it is used for all purposes of construction, and can be obtained in long, straight sticks or logs, free from knots and many other defects. There exist two apparently distinct species, presenting practically the same outside appearances. The one, however, has a large proportion of sap-wood and a small proportion of heart wood. This should not be used in structures unless immersed under water at all times. The other has very little sap, and long, large straight pieces can be obtained of almost clear heart. Such timber is unsurpassed for use in structures above ground, exposed to alternate wetness and dryness. In middle latitudes a pine grows, called commonly spruce pine; is used freely where it grows, but is not considered equal to either of the above species in strength or durability. Nor is it in such abundance, but grows scattered over the ordinary forests in greater or less quantities, often high up on the sides of mountains, difficult to cut and haul, and is soon exhausted in any particular locality.

36. For many years the pine trees of the South have been drained of their resinous matter yearly, called bleeding or turpentining. This is done by boxing the tree; that is, cutting a bucket-shaped notch near the bottom, and a series of channels or grooves leading into it. The resinous matter drips into this bucket, and is removed at regular intervals. How

far and to what extent this affects the growth, ultimate hardness, strength, and durability of the timber, is not perhaps known. It would certainly seem to affect the tree injuriously in all of the above respects, and it is not uncommon to specify that no "bled" timber shall be used in trestles, bridge timbers, or other structure above ground. The owners of the forests stoutly maintain that it does not hurt or injure the timber, and the saw-mill owners generally side with them. Both are interested parties, as the one gets a double profit from the standing trees, and the second from the sawing. But whether it injures the timber or not is of but little practical value, as the "bleeding" is almost universal, and it is almost impossible to make large contracts under this restriction; if you do, there is no means of detecting the difference between the two; as a consequence, you may have to pay high for the requirement, and, after all, only get the bled timber. Some carpenters or experts say that they can distinguish the difference. Some recent experiments have been made, and it has been reported that the bled timber is equal, if not superior, to the unbled; but this can only be fully settled after years of experiment.

37. Oak timber flourishes, apparently, in any country or section of the country, and possesses great hardness and strength, as well as durability, but does not attain the same height; is not as straight, is rarely free from knots of large sizes, and cannot be obtained in as long, straight pieces as the pines, but is largely used in many parts of the country. Its properties depend largely upon the nature of the soil in which it grows, that which grows in low or swampy lands being far inferior to that grown in the same latitude and same climate, but on the higher grounds. The first is soft and soppy, and has not the strength or durability of the second, and should not be used above the ground. Post oak, so called, seems to be a small white oak, but is considered even harder and stronger than the white oak; but does not attain as great size, and is used principally for ties and other structures requiring small cross-sections and lengths. Of the several kinds of oak, white,

red, black, post, and live oak, the white and post oaks are alone used in ordinary structures. The live-oak is the hardest, strongest, and most durable timber in the United States; is owned and reserved by the Government, and is only used in ship-building and other governmental structures.

38. Cypress timber grows to a great extent in the Southern States; is durable, and exceeds even the pines in this respect, as well as in the lengths and diameters of the trees. It has not the strength of the other timbers mentioned, but is often used for piles and for the various parts of the trestles, but in somewhat larger dimensions when required to carry the same loads. Owing to the ease with which it splits into thin slabs, and its remarkable durability when exposed, it is used to a large extent in making shingles, staves, weather-boarding, and the like. There is a species of this timber called black cypress, which has a durability equal to that of any timber. It is, however, rather scarce, and is rather small in diameter, but is much sought after for the piles of trestles in the Southern swamps. It is certainly far superior to the pines in durability.

39. The writer does not mention the chestnut, poplar, elm, cedar, etc., as these are seldom found in such structures as are embraced in this work, though possessing many valuable and useful properties, and are largely used for many purposes requiring durability, ease of working, and where no great strength is required.

40. There are a few general principles which will be of service in determining the general properties of the various species of timber trees.

1. The heaviness generally indicates good timber, as does also the darkness in color. 2. The slowness of growth, as indicated by the narrowness and closeness of the annual rings. 3. The larger the proportion of heart timber, which is generally distinguished from the sap-wood by a darker color, and a harder, firmer material, the better the timber. 4. All timber grown on a sandy, elevated, well-drained area is superior to that grown on very low and swampy ground. 5. Of the same species, that grown in a cold climate is generally considered the

best, whereas of different species the best is grown in a warm climate. But, fortunately, such timbers as grow in this country, whether North or South, and on almost any kind of soil, except a very swampy soil, that can be obtained in the requisite lengths and sizes for large, heavy structures, have the necessary strength, and are not materially different in point of durability. Our choice is largely controlled by circumstances.

ARTICLE XXXVII.

DURABILITY OF TIMBER.

41. THE average life of timber when exposed will rarely exceed from eight to ten years, and unless covered or preserved by artificial means of some kind, all structures should be entirely renewed in that time. The same timbers under apparently the same conditions are very variable in point of durability,—this may arise from one or more causes, which will be alluded to in another paragraph,—and a structure will begin to show decided evidence of decay in some of its parts in from two to four years. This can only be discovered by careful and continuous inspection. The renewal should therefore commence at an early day and be continued as the case demands until entirely renewed, and, in fact, renewal and repairs should be practically continuous. It is due to the neglect to do this that many of our most serious accidents can be traced. The bulk of the timber used on railroads is consumed in building trestles, for both rapidity of construction and decrease of first cost often prove the impracticability of obtaining, on the first construction of a road, a more durable material. Therefore on almost all new roads many very long and very high trestles are constructed. These are called and regarded as temporary structures, and are frequently not built with as much care and strength as they would be if intended to remain permanently as timber structures. Financial considerations prevent the substitution of earthen embankments, masonry arches or abutments, or

iron viaducts,—this is intended to be done from year to year,—and the old trestles must be maintained at a minimum cost for repairs. Rotten stringers and posts are patched up by spiking or bolting strips or planks to them, not unfrequently doing more harm than good. Heavier engines and trains are run over them, taking the risk, until finally the structure collapses, resulting in great loss of life or property ; and so-called experts are called in to explain the causes of the disaster. These often satisfy courts and juries, but, in fact, the structure has simply collapsed from inherent weakness on account of rot.

42. Weakness resulting from rot generally arises at the joints, where timber rests on timber—as the surfaces of contact between tie and stringer, between stringer and cap, cap and post, post and sill. The deterioration does not take place first where it can be seen, but well in and under the top piece. The entire stringer may show a hard, firm, sound surface, yet in unseen parts it may be rotten to the core. (Simple knocking on the surface does not always indicate the exact condition of the interior. The writer has seen this tried many times, and erroneous reports made thereon. The only satisfactory test is to bore into the timber with a gimlet or auger, the bit not exceeding $\frac{1}{4}$ inch diameter.) It is easy to understand why this is the case. The moisture on an exposed surface evaporates rapidly under the influence of the sun and wind, whereas that portion which lodges in the joints between the timbers remains for a longer time, and if it has access to the ends of the timber it will spread for a considerable distance in the direction of the length of the stick. The moisture and internal heat combined with all the conditions favorable to decay, rot will take place in the interior of the stick and under the top stick. This is the condition with the ends of the stringer, the top and bottom of the posts, and to a less degree under the ties. Stringers may be perfectly sound in the middle of their lengths, and rotted to a dangerous degree at the ends, especially on the under side. The same in the posts. The writer has examined miles upon miles of old trestles, boring into the timbers at the ends and at one or two

intermediate points, and rarely has he observed any variation from the above rule. The mortise-and-tenon joint is particularly favorable to bring about the above injurious conditions. This will be further alluded to in discussing the subject of joints. On examining an old structure, bore into the ends of the stringers, the top and bottom of the posts, and obliquely under the ties at the middle of the stringer. If no signs of decay are thus discovered you may be satisfied that the rest of the piece is sound, unless unseen defects existed in the beginning, which generally should be seen on the outside; and this timber should have been originally condemned. Oak timber sometimes, when standing, is rotten in the centre, but this will generally be discovered in cutting down the tree, or in the subsequent sawing of the log.

Article XXXVIII.

MEANS OF PRESERVING TIMBER.

43. TIMBER should be free from such defects as cracks radiating from the centre, or cracks which separate one annual layer from another, splitting as it were into rings, or upsets where the fibres have been injured by compression, or large knots. Doty or spongy places indicate incipient decay. Almost all timber will show cracks when exposed, after being cut, in the green state, to the sun or high winds. If these only extend to a short distance into the timber, and for short distances in the direction of its length, no especial notice need be taken of them, for the cracks are, in fact, unavoidable; but if they extend half the way or all of the way through a stick, it should be condemned, as they materially weaken the timber, and in addition will aid in hastening rot. Large knots also weaken the timber, as they do not adhere strongly to the surrounding fibres, and in addition the fibres themselves around the knots are upset or crippled. The knots also in time frequently become loose and separate from the surrounding timber. A limited number of small knots, if not occurring in

the same vertical or horizontal plane, or not extending entirely through the stick, need not be a serious objection. A little sap on the corners of large sticks has practically the effect of reducing the area in proportion to the amount of sap, as the sap rapidly rots. It is almost impossible to secure large-sized timbers in great quantities entirely free from all of the above defects, but the right to limit the defects should be reserved by the engineer. This is done by specifying that the timber shall be clear-heart, and free from any of the above defects. The material must be either condemned at the mills or on delivery at the site of the work.

44. The best method of preserving timber is by natural seasoning; this is done by exposing the timber to air in a dry place, but protected from hot suns or high winds. This is a slow process and requires many years; the time may be lessened by first soaking the timber in water, as this will dissolve a portion of the sap, and the drying will be more rapid. In artificial seasoning the timber is exposed in a close building to a current of hot air, the time required depending upon the dimensions of the timber. In both cases the timber should be piled with such intervals as will allow free contact of the air with its entire surface, the object of all seasoning being to extract the sap from the pores of the timber. The immense demand for timber has practically rendered natural seasoning impossible, and nearly all timber used in floors, doors, window-frames, mouldings of houses, etc., is artificially seasoned. But for the large structures on railroads, and similar structures, the timber has little or no seasoning, and where great durability is required some artificial method of preserving the timber must be resorted to. But it is useless to resort to any of these means unless the sap is first removed, as the object of seasoning is to remove the sap, so the mode of preserving the timber is to keep the moisture from again entering the pores, by filling them either entirely or partly at and near the exposed surfaces with a durable and impervious substance.

45. For wooden bridges a very excellent method is to entirely enclose the bridge in a sheeting of plank and a tin or

shingle roof; by this means the timber has time to be naturally seasoned at the same time that the structure is being used. The timber in such cases will last for many years; cracks will be prevented, and if at the expiration of one or two years the timbers of the bridge are painted and kept well covered with paint, the structure will last for a great length of time. But the practice of painting timber before the sap is entirely removed is to be condemned, as it does more harm than good; it preserves the timber from decay on the surface, but hastens the rot on the inside. Builders of wooden bridges are always in a hurry to paint the timbers, as it tends to prevent cracks before the structure is completed; but cracks are the lesser evil of the two. Timbers of bridges should not be painted when fresh from the mills. The red paints are almost exclusively used; but these differ materially in durability, and only the standard brands should be used. Oil paints in any colors can now be purchased in cans or barrels, and ready-mixed for use. A coating of pitch or tar is also used as a paint; it is unsightly, but gives good results.

46. Other artificial means of preserving timber consist in filling the pores of the timber with solutions of metallic salts, such as copperas or sulphate of iron, corrosive sublimate or bichloride of mercury, chloride of zinc, sulphate of copper. The timber can be saturated with these salts after the sap has been expelled, or forcing them into the pores under pressure, driving the sap ahead. All of these substances seem to preserve the timber as long as they remain in the pores; but they are gradually dissolved and removed by water. Owing to the expense and time required in the application of these they are but little used in this country. Owing to the abundance and cheapness of good timber, it is found more economical to let the timber rot and renew the structure from time to time. Creosote, or the heavy oil of tar, has, however, been used to a considerable extent in this country, and it is claimed that, although it will materially increase the first cost of the structure, it will ultimately prove economical, as it adds greatly to the durability of the timber, which lasts three or four times as

long as timber not creosoted. The sap and moisture are first exhausted by creating a partial vacuum in an air-tight vessel or tank, and then forcing the creosote into the pores of the timber under a heavy pressure. This is a rather expensive process, making the cost of the structure from 2 to $2\frac{1}{2}$ times as much as the natural timber, and for the reason above stated it is used to only a limited extent. If it were universally used, the cost might be materially reduced. As it is, the application is mainly confined to the treatment of piles and timber used in sea-water; here it becomes a necessity, as timber is rapidly destroyed by sea-worms. The Teredo navalis enters the timber no larger than the point of a pin, and eats towards the centre, growing as it progresses, reaching the size of a grub-worm. Thousands of these worms attack the timber, completely honeycombing it, and destroying it in less than a year. It is difficult to detect the infinitesimal holes on the surface. Large piles have to be renewed in a year after immersion. These worms, however, only eat between the low-water line and the bed of the stream. It has, therefore, been suggested to cover the piles with a sheeting of copper or copper tacks, or to fasten a hollow cast-iron cylinder to the top of the pile, and drive the pile by means of blocks of wood resting on top of the iron, to receive the blow of the hammer, or to place a follower, a short pile in the cylinder, resting on top of the pile, and reaching above the cylinder in order to transmit the effect of the blow. Iron, however, stands but poorly the corrosive effect of sea-water; none of these have yet been proved to give economical and satisfactory results. Creosote is, therefore, commonly resorted to.*

47. The durability of timber is materially affected by the time of cutting down the trees; this should be done generally in the winter, when the sap is not running. Ordinarily, little attention is paid to this, and the trees are cut down and carried to the mill when needed, regardless of the time of the year.

* So-called "vulcanized timber" is now produced by subjecting green timber to a great pressure and high temperature, which converts the sap and other deleterious fluids into antiseptic compounds, which become more or less solid and impervious to water. Experiments indicate an increased strength, stiffness and durability.

The age at which trees are cut is also a matter of much importance. Owners of forests desire to get rid of the older trees, and timber from trees that show evident signs of deterioration from age is often forced on the engineer. Such timber is brittle, sometimes soft, spongy and "doty"; the latter can be easily detected by its appearance, and it will clog the saw while cutting through it; such timber has really commenced to decay, and will rapidly rot when exposed. Often a stick of timber will show this "dotiness" at one end; the other end may be perfectly sound, apparently, and even cutting off a foot or two, the entire stick may seem sound and have a better appearance than many other pieces It is hard to condemn such timber, and it is often, if not generally used. It is a safe rule, however, to condemn, as it evidently indicates a defective tree.

48. All kinds of timber constantly immersed in water will last indefinitely. Timber exposed to alternate moisture and dryness will decay rapidly, as also when moist, and in a warm or confined air. Timber exposed to confined air alone decays by what is known as "dry rot," and crumbles into a powder.

49. As before mentioned, in exposed structures, such as trestles, the timber rots first at certain well-determined points, as the joints in framed structures, and while it is expensive and impracticable to undertake to preserve the timber of such structures by any of the processes above mentioned, it is easy and comparatively inexpensive to prolong the life of these timbers at the joints, and thereby the life of the entire structure, as the strength and durability of the weakest part measures that of the whole structure. This is done simply by painting all surfaces of contact between timbers, as the top and bottom of every post, the ends of the stringers, with hot asphalt or coal-tar, either alone or properly thickened with lime, applied just before putting the parts together; this is simple, cheap, easily and rapidly applied. To include this item in the contract would cause an exaggerated demand for increase in price. It would be carelessly executed and often neglected altogether; but even under these circumstances it would be a great improvement on the ordinary plan. But it would pay the com-

pany to employ a man whose entire time should be devoted to a faithful discharge of this duty, and to purchase the necessary materials. Three fourths of the timber removed from old structures is often as sound as when first used; and, if this simple remedy will preserve the remaining one fourth, it should certainly be adopted.

ARTICLE XXXIX.

TIMBER TRESTLES—(CONTINUED).

50. THERE are two other types of trestle possessing several advantages, but seem to be seldom used; these will be briefly described. In the first, whether the trestle is one or more stories high, the posts are made of several small pieces of timber bolted together, instead of one piece, as in the ordinary trestle. Four pieces, 6 ins. × 6 ins., give an area of 144 square inches, the same as one piece 12 ins. × 12 ins. These pieces bolted together with packing blocks between give stiffness to the columns by increasing the ratio of the least diameter to length; cross and longitudinal pieces can be placed between the pieces, making thereby in some respects a better and stronger connection between all of the main members of the structure, and with an increased number of bolts; pieces and members can be built into a structure possessing great stiffness and strength. This form of trestle has been used in very high trestles and timber piers. The columns may contain any number of the small pieces bolted together, depending upon the height of the trestles and the length of spans to be carried.

51. The other type differs slightly from any of those above described. The writer has never seen it constructed, but it seems to possess the advantage of bringing each member to bear a part of the load at the same time that it gives spread to the base and increased lateral stiffness; it can be better understood by conceiving each story first, as framed for a single story trestle; set one on the top of the other, and each successive story to have one additional batter-post on each side of the centre parallel to the batter-post in that section, but placed so

as to be in the prolongation of the batter-posts in the sections above; the caps and sills of each section being lengthened to suit the above conditions, the usual longitudinal and diagonal braces being used. In this construction, the outside batter-posts extend in one continuous line from top to bottom, the next from the bottom of the first to the bottom, and so on. Distance between vertical posts gradually and uniformly diminishes from top to bottom. This avoids a sort of haphazard placing of the additional pieces in the lower sections, which often exists, and sometimes looks as if pieces of timber were simply inserted to fill up vacant spaces, without any regard to their forming any integral part of the structure itself. And in fact, the strength of the timbers exceeds so many times the strains brought upon them that not much pains is expended upon placing them in such a manner as to be of any great advantage. (See Fig. 32, (*a*), (*b*), and (*c*).)

52. In trestles especially, the joints are of importance, as the structure is generally built of green timber entirely; is exposed to all conditions of weather without any protection of any kind, and is generally run over at high speed, causing constant hammering and vibration, if at all loose. The repairs are made in detail and constantly, without stopping the trains. One piece is taken out at a time and a new piece inserted. Some forms of joints are more liable to rot than others; some reduce the effective bearing surfaces more than others; some make it more difficult to remove a piece than others. Therefore that form of joint which offers the least number of the above objections, due regard being had to the strength of the structure, should be adopted.

ARTICLE XL.

JOINTS AND FASTENINGS.

53. THE principal joint in trestles connects a strut and a tie or a strut and a beam. The mortise and tenon joint is seldom used. (See Fig. 33, (1) and (5).) This joint is formed by cutting a rectangular hole in the face of one of the timbers, about 8 or 8½ ins. long, 3 to 3½ ins. broad, and about 6 ins.

deep, and cutting on the end of the other piece a tenon, theoretically of the above dimensions, but practically from $\frac{1}{4}$ to $\frac{1}{2}$ in. smaller in each dimension. This tenon is inserted into the mortise, and in a hole from $1\frac{1}{4}$ to $1\frac{1}{2}$ ins. bored through the tenon an oak pin or treenail is driven; this constitutes the connection. In this joint from 24 to 27 sq. ins. of bearing surface are lost at the centre of the post, as the tenon never fills accurately and fully the mortise. The bearing surface of the post consists in a narrow shoulder around the tenon. Not unfrequently the post has sap on the corners, extending often on the face of timber for several inches, practically reducing the bearing surface in proportion to the amount of sap. The mortise forms a receptacle for water; sometimes a hole is bored in the bottom of the mortise entirely through the timber. This prevents any great accumulation of water, and affords also slight ventilation; but nevertheless water enters, and remains for a greater or less time. The tenon only serves to hold the pieces together. This joint, therefore, reduces the strength of the pieces, increases the tendency and rapidity of decay, the strength of the joint itself depending upon a small projection of timber in conditions most favorable for rot. Yet it is used more than any other joint. The hot tar and lime paint would certainly be of very great advantage in this case. It is difficult to insert a new timber, when a rotten one is removed, without lifting the entire cap. The cost of framing is somewhat increased in cutting the mortise-and-tenon.

54. The writer has for the above reasons used to a great extent the joint shown in Fig. 33, (2). This joint is formed by simply cutting a notch or dap about 1 in. deep entirely across one of the pieces; the end of the other is simply cut square, fitting into the dap. By this means the entire strength of both pieces is available, moisture is less apt to enter and remain, a new piece is more readily inserted. The rotting of the sap on the outside of the timber affects the strength of the post but little in proportion, as the harder and stronger heart remains. The post can be slightly chamfered at the top, so that the cap can slightly project over it, materially aiding in

excluding the water. The pieces are held together either by drift-bolts, or preferably by straps and bolts, drift-bolts rendering repairs very difficult. Timber strips can be used instead of iron straps, but this increases the tendency to rot, and makes a bungling-looking joint. It would seem that this joint would possess in every respect a great advantage over the mortise-and-tenon joint.

55. A dovetail-joint, or a joint formed by halving the timbers into each other, as in Fig. 33, (3), and bolted together, forms a good connection, but is rarely used in trestle construction; yet it is the usual joint used where both of the pieces are horizontal, particularly in cribs for holding concrete or broken stone, and in the walls of crib coffer-dams, as the pieces act both as a strut and tie brace.

56. The caps and sills are sometimes made in two pieces 6 ins. × 12 ins., instead of one piece 12 ins. × 12 ins.; these pieces placed an inch or an inch and a half apart; daps of sufficient depth cut into these, so that the tenon can enter between them; the pieces bolted together between the posts and also through the tenon. This possesses the following advantages: The tenon has good ventilation and does not rot so rapidly; the cap and sill are likewise preserved; new caps, sills, and posts can be inserted with ease.

57. The inclined or batter posts of trestles are connected in the same manner as the vertical post. When the mortise-and-tenon joint is used, it is so cut that the tenon stands in a vertical direction when connected, and not in the prolongation of the axis of the piece, as seen in Fig. 33, (5); or it can be made as seen in Fig. 33, (6). In these cases the tenon or shoulder should be at least 2 feet from the end of the piece in the direction of the strain, so as to prevent shearing off the timber. If the posts are very much inclined, this tendency should be resisted by bolts as shown in Fig. 33, (6) and (7). This is rarely ever necessary or used in trestle construction.

58. Longitudinal bracing is not generally used in a single-story trestle, unless the trestle is quite high; but is always used

in a trestle of two or more stories. It is a good and safe rule, however, to use it whenever the trestle is over 10 or 12 ft. high. A continuous course of 3-in. plank, spiked or bolted to the outside posts, will add greatly to the stiffness of the trestle, and should always be used if the trestle is constructed on a grade or incline. If spiked to the top of one bent and the bottom of the next in the direction of the descending grade, it will be more effective than if placed horizontally. For higher trestles it is 6 × 6 in. or 8 in. timber, and is bolted to all posts, at the top of each section. Diagonal longitudinal bracing is also sometimes used. The horizontal bracing at the division between the sections or stories is generally notched over the caps as well as bolted to them.

59. Besides the above-mentioned joints, which are principally used in connecting pieces that make either acute or right angles with each other, there are others for lengthening ties or struts. In the case of struts which transmit a compressive strain, all that is necessary for this purpose is to bring the ends square together. But unless fastened in position, there is danger of one piece slipping on the other. This can be prevented by the simple fish-joint, which consists in spiking or bolting strips of wood or iron straps to the faces of the pieces (see Fig. 33, (12), (13), and (14), or by halving the pieces together and holding them in position by bolts or straps, or by a combination of these (see Fig. 33, (10) and (14)). When the pieces overlap and are bolted together the joint is called a scarf joint. Any of these joints shown in the above-mentioned drawings will serve to lengthen struts. But as the strain on the bolts, straps, or fish-plates is small, the simplest connection is the best and the cheapest. Iron sockets are used also in many cases, the ends of the pieces being simply inserted into the open space, and may or may not be bolted; or holes may be bored into the ends of the pieces, and an iron pin inserted in one piece and projecting out, the other piece being then placed on top. This makes a rather weak connection if there is any tendency to bend. Round timbers, such as piles, can be lengthened by halving the pieces, and when placed together drive iron rings

on tightly so as to bind or clasp the pieces together. This is perhaps the best method of splicing piles.

60. To join ties together, any of the joints (Fig. 33, (10), (11), (12), (13), and (14)) answers the purpose; that is, either the fish or scarf joint, or both. It must be remembered in this joint that as the tendency is to pull the pieces apart, the connections or fastenings have the entire strain to bear, and should therefore be as strong as the parts connected, after deducting for bolt-holes, indents, etc. We will describe each joint briefly. Fig. 12 is a simple fish-joint. The aggregate strength of the fish-plates A must be the same as that of the uncut pieces; their length must be such that when under maximum tension the bolts must not shear or cut out to the ends of either the fish-plates or the main members; or the area sheared or split, multiplied by the resistance to shearing must, be equal to the area of the ties multiplied by the resistance to tearing, also to the sum of the areas of all the bolts multiplied by the resistance of wrought-iron to shearing across; or in symbols, $F \times s = T \times t = B \times i$, in which $F =$ equal area of timber liable to shearing, $s =$ safe resistance to shearing per unit of area; $T =$ to effective cross-section of ties, i.e., after deducting bolt-holes, indents, etc.; $t =$ safe tensile strength per unit of area; $B =$ sum of areas of bolts; $i =$ resistance to shearing of iron per unit of area. Assuming unit of area as 1 sq. in., and area of ties as 144 sq. ins. (12×12 in. sticks), and substituting the average units of strength, we have $F \times 400 = 144 \times 1000 = B \times 40{,}000$, hence $F = 360$ sq. ins., or in a stick 12 ins. deep a single bolt should be at least 2.5 ft. from the end of the timbers, or two bolts in the same distance, one 1.25 ft. from the end; and $B = 3.6$ sq. in., or two bolts $1\frac{1}{2}$ in. diameter. This gives the least allowable values for F and B, as the least unit values for tensile strength and the greatest unit values for shearing, both for iron and timber, is used. In this case both fish-plates and bolts have the entire strain to bear, each on its own account.

61. In Fig. 33 (13) the fish-plates are indented into the ties; the effective area of the tie is therefore reduced to that extent, and the strength of the connections is increased as additional areas are presented to resist shearing; therefore the connections need not be as strong as in the first case. The principle of determining the number of the bolts and their position, as also area of fish-plate to be sheared, are the same as above. But this joint sacrifices the strength of the main tie, and causes great waste of material. Both joints present a bungling appearance. Iron bars or straps answer the same purpose and look much better, and in permanent structures are ultimately more economical.

62. In the simple scarf-joint, Fig. 33 (10), the strength of the joint depends entirely upon the resistance to shearing of the timber and the iron bolts; the effective area of the tie is reduced to one half; consequently 50 per cent of the timber is wasted, in addition to the waste in the overlap, each stick being from 4 to 8 ft. longer than actually required in the fish-joint. In these joints hardwood keys are introduced, as shown in the drawings; these being driven between the ties, projecting an inch or two into both, increase the areas to be sheared, the bolts serving mainly to bring the parts into close contact. Fig. 33 (14) shows a combination scarf and iron fish-joint, with bolts and keys. It does not seem to possess any advantage over the plain fish-joint with iron straps, and is more difficult to frame. Keys should always be placed horizontally; if vertical, water easily enters, and causes rot. Fig. 33 (11) shows a scarf-joint which will hold without bolts; only one third of the strength of the timber is secured, and unless the timber is thoroughly seasoned, and the framing carefully executed, there is but little strength in the joint; bolts should always be used in all joints to resist a tensile strain, whether fish or scarf joints.

63. Timber has a greater resistance to tearing than it has to crushing, but owing to the difficulty of connecting several pieces to resist tensile strain, as seen above, without great waste of material, the general practice is to use timber for members under compressive strain and iron for those members under tensile strain, unless single sticks can be secured of sufficient length. In Howe-truss bridges for railways, and timber highway bridges, the bottom chords are made of timber. In the

first iron fish-bars are almost always used at the joints, in the second wooden fish-plates with indents or projections: and in addition the chords are made of three or more pieces thoroughly bolted together at short intervals, with iron or wooden packing-blocks between.

64. Fig. 33, (8) and (9), shows the construction of king and queen post roof or bridge trusses for spans from 20 to 40 ft. in length. The lower horizontal member is called the tie-beam and is under tension; the upper is the straining beam and is under compression. The end inclined members are struts under compression. The verticals are ties under tensile strain; the one on the left is a single piece resting on top of the tie-beam and connected with it by a stirrup, which is simply an iron strap, or bar bent at right angles at the ends, passed up under the tie-beam and held by bolts to the vertical. The one on the right is made in two pieces, the tie-beam passing between them and resting on shoulders cut in them from 1 to 2 ft. from the lower end, the two pieces held together by bolts, packing-blocks being placed between where necessary. This is an over-trussed or through span. Fig. 32, (*b*) and (*c*), represents the under-trussed or deck span constructed for the same purpose. This is used in trestle-work when the spans are from 20 to 25 ft. from bent to bent. Instead of these trussed beams, timber built beams are sometimes used. Fig. 33 (1*a*, 2*a*, 4*a*, 5*a*, 6*a*) shows this construction, in which 2, 3, or 4 pieces of timber are built together with bolts and keys. Two pieces 12 × 12 ins., one on the top of the other, will answer for spans from 12 to 15 ft. long; two pieces side by side, with a third on top, for spans from 15 to 20 ft; and four pieces for spans from 20 to 25 feet. It is evident that there is a great waste of timber, and it is badly distributed to meet the required conditions of strain. Single pieces properly trussed would be more sightly and more economical. Beams are built also, either as straight or curved beams, of plank laid flatwise and bolted or spiked together, and are frequently used in bridges and roofs; their strength is materially less than a solid beam of the same cross-section, probably not more than from

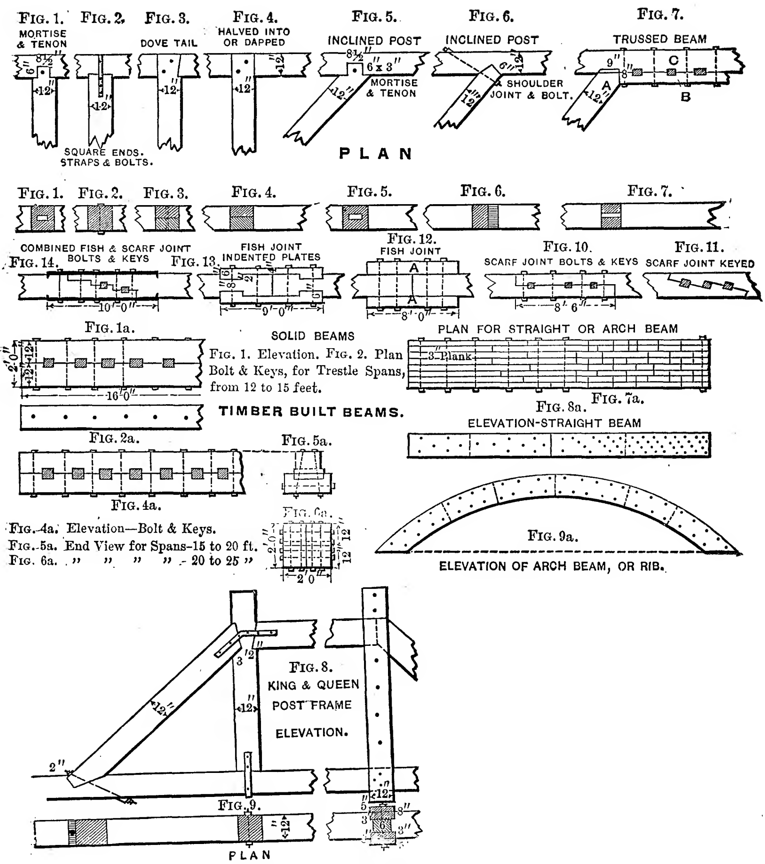


FIG. 33.—DETAILS OF JOINTS, BUILT BEAMS, ETC.

[*To face page* 194.]

one third to two thirds as strong. As a rule, they would not be used when large sticks could be secured, except in temporary structures, such as centres for arches (Fig. 33, *7a*, *8a*, *9a*). From the above description, there are certain general principles applicable in constructing all joints.

1. The joints should be so cut, and the fastenings so proportioned and placed, as to weaken the main members as little as possible.

2. The strength of the fastenings, bolts, straps, and fish-plates, either singly or together, should be equal to the effective strength of the parts connected, and their areas should be inversely proportional to their unit strength.

3. All surfaces in contact should fit exactly throughout, and the direction of the planes of such surfaces should be perpendicular to the direction of strains, and their areas should be sufficiently large to keep the unit strain of that particular kind within safe limits, allowing a factor of safety of 4 in case of steady and of 10 in case of moving loads.

4. All joints should be so arranged and placed as to preclude, as far as possible, access to water, and should be painted with some substance impervious to water.

5. The joints will in general be the weakest part of the structure, and should be taken as the measure of the strength of the whole.

6. In all built or packed beams the parts composing the beam should break joints as far as possible.

7. Where joints are formed by indents, shoulders, or tenons, and held together by bolts or straps, each should have sufficient strength to transmit the entire strain, as from shrinking of timber, loosening of bolts, or other causes, either may have to bear the entire strain. This is applicable in all cases, except in lengthening struts, unless constant inspection is made to see that all parts are properly adjusted.

8. Although there is always a consideration, admissible, of the frictional resistances between surfaces in contact, this should not be relied upon to any but a very limited extent, if at all.

ARTICLE XLI.

SUPPORTS FOR TRESTLES.

65. Trestles are supported in three ways: 1. By mud-sills; 2. By masonry pedestals; 3. By piles.

If any pretext can be offered the first is always adopted; they consist of from 4 to 6 pieces of timber, of any size convenient, generally 4 or 5 ft. long, placed under the bottom sill, either wholly or partly imbedded in the ground, shallow trenches being excavated to receive them. With no precautions taken to provide drainage, and regardless of the nature of the soil, naturally water collects in the trenches, converts the soil into mud, the motion of the train produces a churning action, the trestle rises and falls, gets out of line and level, is then adjusted by driving shingles, thin strips of plank, or anything that can be procured under the sills, and this is repeated until these strips or shims are piled one on the top of the other for 6 or 8 ins. or more, sometimes only under one or two of the posts. Under such circumstances the wonder is that accidents are not many times more frequent than they are. Occasionally engineers require a double row of sills to be laid first and the mud-sills placed across and at right angles to these. This serves several purposes: it may place the foundation-bed below the injurious action of frost; it increases the area of bearing surface; it lifts the sill slightly above ground, permitting ventilation and consequently preserving the timber; it is certainly not very expensive. It should always be required.

2. When the nature of the ground will admit of mud-sill under trestles, it will generally be found that rubble-stone can be obtained at a small cost. Small rubble pedestals should then be built, in pits from 1½ to 2 ft. deep and extending from 6 ins. to 1 ft. above ground. A single pedestal 2 to 2½ ft. square under each post will be sufficient. This is as much superior to the last method as that is to the simple mud-sill.

3. Piles are used when the ground is very soft for any great

depth below the surface, or in swamps. These can be cut off below the moisture line or above the surface. One pile is driven so as to be under each post of the trestle; the sill rests on the piles, and is fastened to them by mortise and tenon, by drift-bolts, or by straps. Commonly where piles are necessary, the trestles are comparatively of small height, the piles reach well above ground, and the stringers rest directly upon them. In such cases the structure becomes distinctively a pile trestle, as distinguished from a framed trestle, and will be explained under that head.

66. Framed trestles may be divided into four classes or types presenting some marked differences in construction.

1. That in which the principal members are vertical, except the outside batter-posts, inclined braces of smaller cross-section being introduced to give steadiness and stiffness. This is probably more generally used than any other. Fig. 31, (*a*), represents a single story or section.

2. The M trestle, in which all of the main members are inclined, and verticals introduced only in the very high trestles, auxiliary inclined braces of smaller cross-section being also used. See Fig. 31 for single story, and Figs. 34 and 34 (*a*) for two or more stories, with details of important parts.

3. The trestle in which the columns are built up of pieces of small cross-section, instead of single pieces of larger cross-section. It can be put together as in either of the above types.

4. That form in which two vertical columns extend from top to bottom, inclined members are used to bear a portion of the load, and to act as main members and braces at the same time. See Figs. 32 and 32 (*a*). This form of trestle has never been used to the writer's knowledge, but it certainly seems to be as strong as, if not stronger than, the other trestles in common use. It will be noticed that straps are used, instead of the mortise-and-tenon joints. The writer believes that they make a better and stronger trestle. It is also shown as resting on masonry pedestals for the same reason. Either of the above forms of trestle is strong enough, and

that one should be adopted which requires the least material, requires the least amount of work in framing and erecting, and is more easily renewed and repaired in whole or in part. Figs. 32 and 32 (*a*) show the method of trussing the stringers by the use of straining pieces and struts, and Fig. 32, (*b*), by the use of iron rods. In the latter case it is better to use two ties or rods, and, to prevent the necessity of boring holes in the string-pieces, the stringer should be made of three pieces, with small intervals between them through which the rods can pass. On the ends of the stringers thick iron washers should be placed, through which the rods pass and upon which the nuts bear, so as to prevent the rods from cutting into or crushing the fibres of the timber at the ends of the stringers. As the spans are 25 feet from bent to bent, the stringers do not break joint on the cap, and unless bolsters or fish-plates are used, the stringers would not have over 5 inches of bearing on the caps, which is not enough. In Fig. 32, (*a*), two 6-inch fish-plates are bolted to the stringers and rest on the cap; by this means the bearing surface is increased and at the same time the stringers are tied together. This the writer prefers to the bolster connection as shown in Fig. 34 under the stringer and bolted to the cap.

67. All of the main members of the trestle-bent are subjected to a compressive or crushing strain, except the bottom and intermediate horizontal pieces or sills; these are subjected to a tensile strain and a crushing strain across the grain by the pressure on the posts. The tensile strain tends to shear the layer of timber between lower end of each post and the end of the sill. To resist this the end of the post should be from $1\frac{1}{2}$ to 2 ft. from the end of the sill, as shown in all of the drawings. As the inclination of the batter posts is small, this strain is small, and consequently bolts are not needed or used. The strength of the timber to crushing transversely, whether pine or oak, is very great; the greatest loads that can possibly come on the posts would make no impression on the timber of the sills between them. Trautwine says that a pressure of 1000 lbs. per square inch will not indent yellow pine or oak

more than the thickness of a sheet of writing paper, and white pine not over $\frac{1}{8}$ of an inch. The writer has subjected soft pine cushions in testing the crushing strength of stone, to a pressure of 5000 lbs. per square inch, with no perceptible effect upon them. The upper cap is under both longitudinal and transverse crushing strain, and always has ample strength. The posts are under longitudinal compression. The lengths of such pieces in proportion to their least dimension is an important factor in their strength. In very short columns the resistance to crushing is simply proportional to the area of the cross-section.

68. The coefficient of resistance to crushing, or the strength per square inch of area, for green timber is about 5000 lbs. per square inch. This, however, decreases in a rapid ratio as the length increases, and when the length is 30 times the diameter or least side it would crush under less than $\frac{1}{2}$ of $5000 = 2500$ lbs. per square inch, and the safe load should not exceed $\frac{1}{5}$ to $\frac{1}{10}$ of this, or from 500 to 250 lbs. Seasoned timber is about twice as strong as the green timber. The usual practice is that the length of the column should not exceed 20 times its least dimension, or a stick 12×12 ins. should not be more than 20 ft. long; so we find that the height of a story or section of a trestle-bent does not exceed 20 to 25 ft. If the bents are $12\frac{1}{2}$ ft. apart, the greatest load per foot of span would be about 6000 lbs. or 75,000 lbs. on each bent of the trestle. This is really supported by four pieces, but two pieces 12×12 in. $= 144$ square inches would bear safely, at the low limit of 250 lbs. per square inch, 72,000 lbs.; but assume that the two batter posts together bear $\frac{1}{3}$ of the load and the two vertical posts $\frac{2}{3}$ of the load, each vertical post would carry only 25,000 lbs., and each batter post 12,500 lbs. $A \times 250 = 25{,}000$, and $A' \times 250 = 12{,}500$; or the area of the vertical posts $= A = 100$ sq. in.; or one piece 10×10 ins. or $12 \times 8\frac{1}{2}$ ins. would be sufficiently large for the vertical posts, and for the batter posts $A' = 50$ sq. ins. or 1 piece $6\frac{1}{2} \times 8$ ins., and in the M trestle each post would carry $\frac{1}{4} \times 75{,}000 = 18{,}750$ lbs. $A = \dfrac{18750}{250} = 75$ sq. ins. and each post

would be $8 \times 9\frac{1}{2}$ or $8\frac{3}{4} \times 8\frac{3}{4}$ ins.; but in either case the dimensions are seldom if ever less than 10×12 ins., and more commonly 12×12 ins. in cross-section.

69. In the case of a 25-ft. span the uniform load per foot of length would not exceed 5000 lbs., or on each bent 125,000. In the M trestle each post would carry 31,250 lbs. its area need not exceed 125 square inches or one piece $12 \times 10\frac{1}{2}$ ins.; and for other forms of trestle, assuming as before one third of the entire load as borne by the batter-posts and two thirds by the vertical posts, the batter-posts would be 10×9 ins. each and the verticals 12×14 ins. In this calculation the column or post is supposed to be at least 30 ft. long, which is rarely the case, the diagonal or X bracing increases its strength materially; so it will be seen that even in 25-ft. spans the verticals need not exceed 12×12 ins. In very high trestles the pressure on the lower posts is increased by the weight of the structure; but as the number of posts is greater in the lower sections, the unit pressure on any one post would never exceed that of the posts in the upper story, and no increase in the area of the posts is necessary in the lower sections above the dimensions given above and shown in the drawings, though the bottom posts are sometimes 12×14 ins.

70. To determine the area of the cross-section of the stringers we will use the formula $mWl = nfbh^2$, as the beam is under a transverse load of 6000 lbs. per lineal foot, and is liable to give way by bending or cross-breaking. The greatest possible load is 6000 lbs. per foot, and the unsupported length of span, being in bents $12\frac{1}{2}$ ft. centres, only $11\frac{1}{2}$ ft. The total uniform load is 69,000 lbs., and the equivalent centre load $= \frac{69,000}{2} = 34,500$ lbs.; and as this is supported by four pieces 6 in. wide or thick each, each piece will only have to bear $\frac{34,500}{4} = 8625$ lbs. Then we have $m = \frac{1}{4}$, $W = 8625$; $l = 11.5 \times 12 = 138$ ins.; $n = \frac{1}{6}$; $f = 1000$; $b = 6$ in. Substituting and finding value of h, we have $\frac{1}{4} \times 8625 \times 138 = \frac{1}{6} \times 1000 \times 6 \times h^2$; $\therefore h^2 = 297.56$; $\therefore h = 17\frac{1}{4}$ ins., the depth of the beam;

from this each stringer should be composed of two pieces $6 \times 17\frac{1}{4}$ ins. This gives a little greater depth than the actual practice, on account of the large factor of safety used, or what is the same thing, the small value of f and the large value given to W. The actual dimensions in practice are 6×14 ins. to 6×15 ins. For longer spans the calculation in every respect would be similar, but, owing to the difficulty of getting good sound sticks over 15 to 16 ins. deep, the usual practice is when necessary to increase the number of stringers, as three or four pieces 6×15 ins. But for spans over 20 or 25 ft. long it is best to truss or brace the stringers; this trussing is equivalent to dividing the length of the span into three parts, as shown in Fig. 33 (8) or into two parts, as in Fig. 32, (*b*) and (*c*). In the first case the spans are only about 8 ft. long, and in the second 12 ft. in a 25-foot span; so it is evident that in this case no increase in the size of the stringers will be required. The struts in the bracing will have to bear the load on only 8 ft. of span or 48,000 lbs., and being four in number, each will have 12,000 lbs., or at 250 lbs. per square inch there will be necessary 48 sq. ins. in each strut, or a single piece 6×8 in. In the second case the tension-rods under each stringer will have to carry a load on 12 ft. of span or 72,000 lbs. and on each stringer 36,000 lbs.; but this passes from the middle through the rods on both sides to the end, the rod has only to bear 18,000 lbs. of load. This produces a pull or tension on the rod equal to the load multiplied by the length of the rod and divided by its vertical reach, or $18{,}000 \times \dfrac{12.25}{2.25} = 99{,}000$ lbs. The tensile strength of iron is about 50,000 lbs. per square inch, and with a factor of safety of 4, 12,500 lbs., there results $\dfrac{99{,}000}{12{,}500} = 8$ sq. ins. nearly, or a single rod 3 in. diameter or 2 rods $2\frac{1}{4}$ in. diameter each. The pull on these rods tends to crush the end of the stringer. Large washers should be used to keep the pressure in safe limits. An iron plate 12×6 in. $=$ 72 sq. ins. would reduce the unit of pressure to about 1400 lbs., which would be safe. Increasing the length of the vertical decreases pull on the rods.

71. The writer has entered into considerable detail, as it shows the principles of calculating the several kinds of strain, determining the sizes of the different members and the proper construction and connection of the parts, so as to keep the unit strains within safe limits, and are equally applicable to bridges, trestles, floors of warehouses, etc. The formula above used is general, easily remembered, and easily applied when the principle of the lever or moments is understood. The amount and distribution of the load and the way in which the beam is supported are known. In the formula $mWl = nfbh^2$, m is a constant depending upon the distribution of the load and the manner of supporting the beam; W is the total load on the beam; l is the clear span or unsupported length of the beam; n varies with the shape of the beam, and for beams of square or rectangular cross-section is equal to $\frac{1}{6}$; b = breadth of beam; h = depth,—all dimensions in inches; f the modulus of rupture, which varies from 10,000 to 5000 lbs.; but only $\frac{1}{5}$ to $\frac{1}{10}$ of these amounts should be used in practice. For a beam supported at one end and loaded at the other, $m = 1$; uniformly loaded, $m = \frac{1}{2}$; supported at both ends and loaded with a single weight at the centre, $m = \frac{1}{4}$; and uniformly loaded, $m = \frac{1}{8}$. These are common and usual conditions. If we know b and h, we can then find W; or knowing W, as in the examples above, we can find h by giving a value to b, or find b assuming h. This will be sufficient for the purpose now considered in this volume. In the case of a joist in a floor, each joist supports the weight on an area of the floor equal to the length of the joist multiplied by the distance from centre to centre of the joists. The same is true for the flooring of a highway bridge. The load is generally taken as that of a closely packed crowd, and estimated at from 50 to 100 lbs. per square foot of area. For a warehouse floor the actual weight of grain, salt, or other material that can be used must be determined, and the allowance per square foot thereby determined in each. It will be observed that the uniform load on a beam has the same effect at the centre as a single load at that point of half of the amount; therefore, making $W = \frac{1}{2}$ of the total uniform load, the formula reduces to $\frac{1}{2}Wl = \frac{1}{6}\frac{fbh^2}{l}$, or $W = \frac{1}{3}\frac{fbh^2}{l}$ for a beam fixed at one end and uniformly loaded. The general formula reduces for the four usual conditions of loading to

$$W = \frac{1}{6}\frac{fbh^2}{l}, \text{ fixed at one end and loaded at the other.} \quad (1)$$

$$W = \frac{1}{3}\frac{fbh^2}{l} \quad \text{" " " " " " uniformly.} \quad (2)$$

$$W = \frac{2}{3}\frac{fbh^2}{l}, \text{ supported at both ends and loaded at centre.} \quad (3)$$

$$W = \frac{4}{3}\frac{fbh^2}{l} \quad \text{" " " " " " uniformly.} \quad (4)$$

The value of f is taken from tables. It is important to note, when using the tables, in what units l, b, and h are expressed, as l is sometimes in inches and at others in feet, and the value of f is given to correspond. In this volume l, b, and h are expressed in inches, and f varies for timber from 250 to 1500 lbs., according to degree of safety required, and in general it will be from $\frac{1}{5}$ to $\frac{1}{10}$ the ultimate resistance. The value of W will be then the safe load.

72. Timber may be subjected to several kinds of strain. 1st. Crushing or compressive. 2d. Tearing or tensile. 3d. Transverse or bending; which may result in breaking across the grain. 4th. Shearing; a cutting or splitting along or across the grain. 5th. Twisting strain or torsion. The coefficient or modulus of resistance is different for each kind of strain, and of course varies with the kind of material. Mr. Rankine gives for oak and pine timber, for either crushing, tearing, or transverse strain, 10,000 lbs. per square inch, as the ultimate resistance. Green timber is not more than half as strong as seasoned timber. The resistance to crushing along the grain is not

more than half to two thirds its tenacity or resistance to tearing. The resistance to shearing along the grain, 600 lbs. per square inch for pine and 2300 lbs. per square inch for oak.

Mr. Trautwine gives the following table of the ultimate strength per square inch. Green timber is inserted at about two thirds that seasoned.*

To resist crushing when seasoned,	6,000	lbs.	per sq.	in.	oak or pine.
" " " " green,	4,200	"	"	"	"
" " tearing " seasoned,	10,000	"	"	"	"
" " " " green,	6,666	"	"	"	"
" " cross-breaking when seasoned,	10,000	"		"	oak.
" " " " green,	6,666	"		"	"
" " " " seasoned,	8,100	"		"	white pine.
" " " " green,	5,400	"		"	"
" " " " seasoned,	9,900	"		"	yellow pine.
" " " " green,	6,600	"		"	"
" " shearing,	750	"		"	oak.
" "	500	"		"	pine.

These being ultimate or breaking strains, the safe strain should not exceed one fifth of these values, and for heavy rolling loads not more than from one eighth to one tenth of these values.

73. The strength to resist tearing is independent of the length of the member, if the joints and connections are properly made and proportioned. But to resist crushing the strength decreases rapidly with the ratio of the length to the least dimension of the member; and when its length is from 20 to 25 times its least dimension, its resistance to crushing is reduced to 2000 or 2500 lbs. per square inch. But in this case a special formula should be used, such as $w = \dfrac{5000}{1 + \dfrac{1}{250}\dfrac{l^2}{d^2}}$. l = length of column in inches, d the least side of the column in inches, and w the ultimate crushing resistance per square inch. We

* If l in formulæ 1, 2, 3, 4 on preceding page, is in feet, the ultimate *resistance to cross-breaking* in this table must be divided by 18 in order to obtain the value of f, which must be further divided by factor of safety to obtain safe load.

may therefore conclude that for perfect safety the following values should not be exceeded, in pounds per square inch:

To resist crushing for a steady load,	1,000 lbs.;	for a rolling load 500 lbs.
" " tearing " " "	1,250 " ;	" " " 700 "
" " cross-breaking for a steady load,	1,250 " ;	" " " 700 "
" " shearing,	185 to 125 lbs.	

BILL OF MATERIAL.

74. Take, for example, a four-story trestle of the M type, span 30 ft. long (which, all things considered, is probably the most economical), mortise-and-tenon joint, total height of bent 100 ft. (See Figs. 34 and 34 (*a*).)

TIMBER.

	2 guard rails	6″ × 6″ × 25′	= 150	ft. B. M.
	25 cross-ties	6″ × 8″ × 10′	= 1,000	" "
	4 stringers	7″ × 15″ × 25′	= 875	" "
	2 bolsters	10″ × 16″ × 6′	= 160	" "
	6 lateral bracing	4″ × 4″ × 9′	= 72	" "
Fourth Story.	1 cap	12″ × 12″ × 10′	= 120	" "
	4 main posts	12″ × 12″ × 17.5′	= 840	" "
	2 × braces	2″ × 10″ × 23′	= 77	" "
	4 longitudinal braces	8″ × 12″ × 33′	= 1,056	" "
	4 struts under stringers	8″ × 12″ × 20′	= 640	" "
	2 straining-pieces	7″ × 12″ × 9′	= 126	" "
Third Story.	1 cap	12″ × 12″ × 17.5′	= 210	" "
	3 main posts	12″ × 12″ × 20.5′	= 738	" "
	2 " braces	8″ × 12″ × 20.5′	= 328	" "
	2 × "	2″ × 12″ × 29.5′	= 118	" "
	8 longitudinal braces	6″ × 12″ × 31′	= 1,488	" "
Second Story.	1 cap	12″ × 12″ × 23.2′	= 279	" "
	4 main posts	12″ × 12″ × 25.5′	= 1,224	" "
	2 " braces	8″ × 12″ × 25.5′	= 408	" "
	2 × "	2″ × 13″ × 33.0′	= 143	" "
	8 longitudinal braces	6″ × 12″ × 31.0′	= 1,488	" "
First Story.	1 cap	12″ × 12″ × 32.0′	= 384	" "
	2 main posts	12″ × 12″ × 34.7′	= 833	" "
	2 " "	12″ × 14″ × 34.0′	= 952	" "
	2 " braces	8″ × 12″ × 34.7′	= 555	" "
	2 × "	2″ × 12″ × 51.5′	= 206	" "
	longitudinal braces			
	bottom sill	12″ × 12″ × 43.8′	= 426	" "
	Total timber		14,896	" "

Iron.

5	bolts for guard rails....	½" × 14"		4 lbs.
40	spikes " " "	10"		22 "
50	" " " "	10"		28 "
8	bolts for stringers......	¾" × 16"	grip.	16 "
8	cast packing spools.....			16 "
4	bolts for bolsters.......	¾" × 25"	"	15 "
6	" " straining pieces	¾" × 27"	"	60 "
8	" " struts.........	¾" × 24"	"	30 "
	" " foot plates.....			
24	" " long. braces...	¾" × 24"	"	90 "
6	" " × braces......	¾" × 16"	"	13 "
4	" " " "	¾" × 14"	"	7 "
8	drift or rag bolts.......	¾" × 20"	"	24 "
				325 "
4	lateral brace rods.......	1" × 6' 2½"	grip,	66 "
	Total iron..........................			391 "

The total iron should also include the weight of nuts and washers. Either cast or wrought washers may be used. Allowing 2 lbs. for head, nut, and washers for each bolt, the aggregate iron would be 555 lbs. Allowing $30 per 1000 for timber framed and 5 cents per lb. for iron, the above bent would cost $475, or per foot $15.83.

75. This calculation has been made on the M form of trestle (see Figs. 34 and 34 (*a*)), which shows elevation, plan, and details. It is probably as light a trestle as would be good practice for spans 25 to 30 feet long and 100 feet high.

76. The bill of material is given purely as an illustration. Any other form of trestle can be similarly calculated, and comparison as to cost made. It is better in approximate estimates to overestimate a little than to underestimate.

77. The writer does not give the extended tables, usually given in books, of the strength of materials, nor the varying results of different experiments on the same material. It has been his sole object to mention those timbers in common and every-day use, that are likely to be used in the kind of structures considered, and only to give those values of the coefficients of strength which seem to be universally accepted as fair working values in actual practice. Extensive tables are given by Rankine, Trautwine, and other authors.

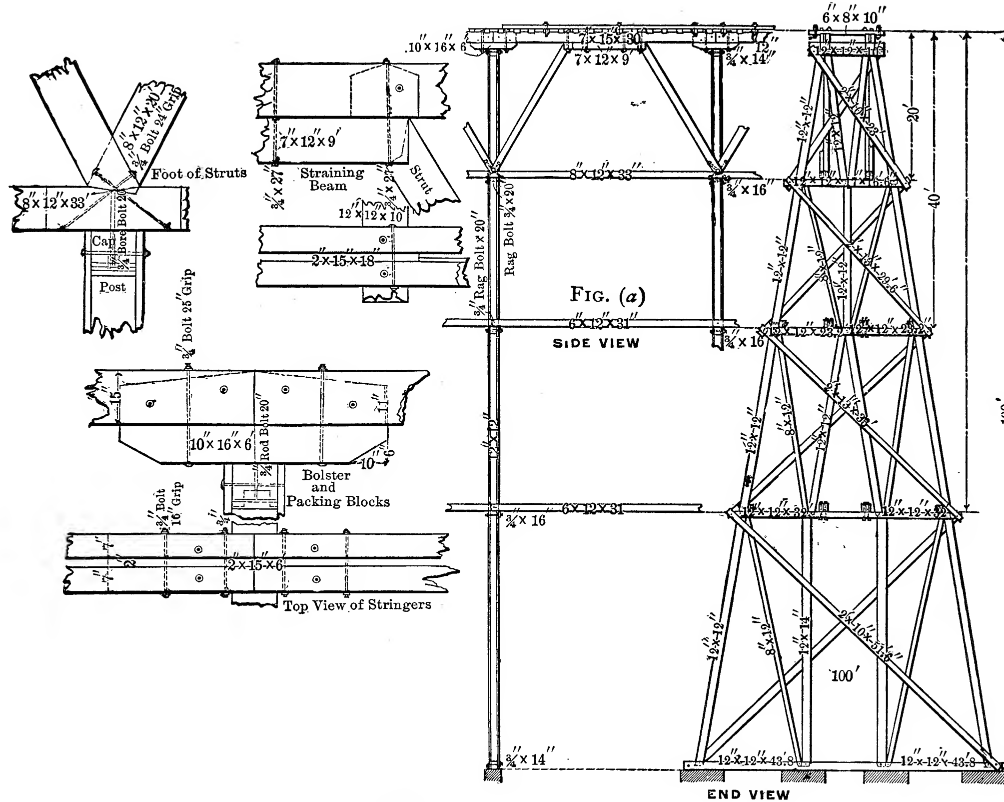


Fig. 34.—Four-story Trestle-bent, Shenandoah Valley Railway. W. W. Coe, Chief Engineer.

[To face page 206.]

ARTICLE XLII.

TIMBER PILES.

78. PILES are used in such materials as are not able to bear the weight of structures, after spreading the base of the structure by the use of concrete or timber, either singly or combined, or where the cost of thus preparing the foundation would be excessive, and also where, although the material is firm enough to bear the weight, there is danger of it being scoured out by the current, thereby undermining and endangering the structure, and often without considerations of the above nature, but purely on account of convenience, expedition, and economy. Piles are either short or long sticks of timber, generally round, sometimes and for special purposes sawed square, or rectangular as in sheet-piles. They are driven into the ground to a greater or less depth, depending on the purpose for which they are used. Oak, pine, cypress, and elm, are the principal trees used for piles. Oak has the advantage of being hard and tough, will stand more hammering, but cannot be obtained as large or as straight and as long as either pine or cypress; is somewhat more expensive in certain localities, mainly on account of the cost of transportation. On account of its heaviness it is apt to sink in water, and large rafts are liable to sink unless buoyed up by some lighter logs intermixed with them, such as poplar. In some localities oak is more abundant than pine, and is consequently largely used. Pine can be obtained in long, large, straight logs, in any lengths up to 90 or 100 feet, and in diameters at the butt end from 12 to 18 inches or more, and from 10 to 12 inches at the small end. The yellow pine of the South is hard and tough; these qualities make it particularly useful for piles, and owing to its great abundance in the South and the fact that it can be floated in large rafts on the many bayous and rivers that flow through the forests, it is comparatively cheap. The same may be said of cypress, but this splits more easily and does not stand the hammer so well. Elm is considered good for the purpose also, and can be found in great abundance in some localities, but does not seem to be used to any great extent when either of the above materials can be found.

79. Piles are prepared for driving by cutting or sawing the large end square, bringing the small end to a blunt point with an axe, the length of bevel being from 1½ to 2 ft. long; and, finally, by stripping it of its bark. This should never be neglected, certainly for that part below the ground. In soft and silty material there is no necessity of pointing the pile at all, and in fact it can be driven in better line when left blunt. A pointed pile on striking a root or any obstruction of the kind will inevitably glance off, and no available power can prevent it; the blunt pile, on the contrary, will cut or break the obstruction; ample experience fully justifies this view. The large end of the pile is chamfered for a few inches from the end, so that a wrought-iron band from 10 to 14 ins. internal diameter will just fit, and will clasp the pile uniformly and tightly with one or two light blows of the hammer. Sometimes a ring from 1 to 1½ ins. less diameter than the pile is simply placed on the top of the pile and driven into it by light blows. This, however, is apt to split long layers from the pile, and in such cases the band is not put on until the pile is more or less battered, and then often very carelessly, and not concentric with the end of the pile. The first method of fitting the ring to the pile seems to be the best. If the end of the pile is not cut square and true, the blow will be received on one edge; this tends to split the pile, to drive it out of line, and break the ring. The band should be made of the best wrought-iron, with metal thickness of at least 1 in. and 3 in. wide, carefully and thoroughly welded; with every precaution the rings will very often break. It is difficult to make foremen put the ring on until the pile begins to show signs of splitting; it is then too late to be of much advantage. They should be required to put them on in the beginning, and if one breaks, require the broomed or battered portion to be cut off and a

new ring put on at once. It may be easy to prevent an initial split, but difficult to prevent it extending when once begun. Unless bar iron is convenient and can be obtained readily, a large number of bands of different diameters should be provided in advance, as rings after heavy and repeated blows will not stand many weldings.

80. In driving piles into hard and compact materials, such as stiff clay, sand, and gravel, the point of the pile is often shod with iron. Unless this is properly done, no great benefit will result, and as commonly done it is of little use. The pile is generally brought to a sharp point; three or four straps of iron are welded together with a sharp point, both inside and out; the end of the pile is then inserted, only touching the straps near the upper ends: bolts are then passed through the straps and piles; often only short spikes are used. Consequently, the bolts split or cut through the timber, until by the force of the blows the pile is made to fit the shoe, or the straps spread; thus more harm is done than good. The only proper way is to have a blunt end to the pile from 4 to 6 ins. in diameter. The shoe should have a solid conical point, the base being of the same diameter as the end of the pile, and should fit it full and true; the straps then extending upon the sides of the piles and bolted to them, the straps and bolts mainly holding the shoe in place, the end of the pile receiving the effect of the blow. Such a shoe will to a great extent prevent the end of the pile from brooming. In such piles as the writer has seen that have been pulled up, he does not recall any case in which the lower end of the pile has split, after hard driving, even without shoes; but the end of the pile would be broomed up to a length of 6 ins. It would be interesting and instructive if the ends of many piles that have driven could be examined. It is rare that piles are ever pulled after being once driven; it is far easier to cut them off or blow them off below the bed of the river, by boring holes and inserting dynamite cartridges. We therefore know but little of the condition of the points of the piles, whether driven with or without shoes.

81. We do know, however, a great deal about the effect of heavy blows on the upper and exposed end of piles, and should be able to learn some important lessons in driving piles from these effects. There is always great danger of piles splitting unless well banded at their tops: and even this does not always prevent it, as piles will often show a split and a consequent buckling below the band and extending to a greater or lesser distance downward; but unless a senseless and useless hammering on a pile is required, a good band well fitted to the end of the pile, and the end cut square, will prevent any serious splitting. The brooming up of the top of the pile will take place as a rule whether a band is used or not; this causes the head to swell and bulge and the bands themselves to tear apart, and occasionally the fibres of the pile are completely crushed below the band. Even when piles are badly broomed, they do not necessarily show any decided or dangerous splits. The head of a pile may broom to a considerable extent without any serious injury to the body of the pile a foot or two below, and when cut off the end of the pile should show a hard, firm, uniform surface. This will only exist where a band has been used. If any great brooming results where a band is not used, splits to a greater or less extent will inevitably exist. The writer has personally superintended 8 or 10 miles of pile-trestling and numbers of pile foundations for piers and abutments in all kinds of material, and has more than casually observed as many more, and now recalls but few instances in which piles have split to any extent when properly banded and banded at the proper time, unless hit many heavy blows after evident refusal to penetrate farther, under a useless law based upon an equally useless formula. As, for instance, that a pile shall not penetrate more than $\frac{1}{4}$ to $\frac{1}{2}$ of an inch by 30 blows of a hammer weighing 2000 lbs. falling from 15 to 25 ft., at each blow, and this without apparently any regard to the depth already in the soil or the rapidity of the blows. The above is a liberal representation of a fact, and piles are hit often from 50 to 100 blows to comply with such requirements, every blow brooming and crushing the head and point of the pile, and splitting and crushing the intermediate portions to an unknown and dangerous extent; the piles often crush between the head and the ground, or under water or under ground, not unfrequently breaking short off. The writer ventures to assert that in all such cases the pile does not move at all; the apparent penetration is simply due to crippling the fibres at some point, generally the head and foot of the pile, but often at intermediate points, the pile supposed to be moving $\frac{1}{60}$ of an inch at a blow. How this infinitesimal distance can be determined or measured in pile-driving seems hard to be understood. Piles can easily be seen to bend perceptibly under heavy blows, and it would require perfect elasticity to recover their exact positions within $\frac{1}{60}$ of an inch, to say nothing of the shortening by brooming or crippling of the fibres. A pile may go from $\frac{1}{4}$ to $\frac{1}{2}$ an in. apparently for each blow in 30, and never actually penetrate the $\frac{1}{100}$ part of an inch.

82. The brooming of the head of the pile has the effect of materially reducing the force of the blow. A pile may apparently have ceased to move under repeated blows of the hammer, but if the broomed end is cut or sawed off, and then struck with the hammer, it may readily penetrate several inches at a blow; but it is hardly ever the case that a broomed end pile is repeatedly cut off, so as to present at all times a hard, firm surface to receive the blow: hence formulæ would seldom be of any practical value in determining the extent to which piles should be driven. There are many formulæ published, but they can scarcely be considered as safe guides in settling the much-disputed point as to the penetration required to bear any definite load, and the results of these under apparently the same conditions are so different that they would only tend to confuse.

83. It might then be asked if formulæ are of no value and no rules as to the penetration allowable in the last 10 to 30 blows. How are we to determine when to stop driving a pile. This is a difficult question to answer directly, for many reasons:

1. It depends to a large extent upon the overlying strata through which the pile has been driven. A long pile driven through a gritty material into a softer underlying strata will

have sufficient frictional resistance to bear with perfect safety the required load ; this resistance being reduced to minimum during the process of driving, but developed to its full extent after an interval of rest. And in the reverse case, that of a firm soil underlying a softer stratum, the ability of a pile to bear a load would be small, notwithstanding the great resistance to farther penetration.

2. In many materials a very great resistance will be developed when the pile has penetrated only a few feet—not a sufficient depth to give steadiness or stability to the structure or to be safe against the effects of a scouring action of the current. The writer drove about 2 miles of pile trestle in an approach to a bridge across the Warrior River, Ala., in a compact sand, and was unable with a 1800-lbs. hammer to drive white oak piles more than from 5 to 6 feet in the soil without battering the piles to pieces. Subsequently this was filled in with earth. A constant vibration in connection with the presence of water would doubtless cause more or less settling of such piles in the long run.

3. Apparently the same material offers a very different resistance to piles driven under the same conditions. In some sands piles cannot be forced over a depth of from 5 to 10 feet ; in others they can be driven from 20 to 30 feet, and again light hammers from 1500 to 2000 lbs. with a high fall will not be as effective in driving piles in stiff clay or sand and gravel, or in a mixed soil, as a heavy hammer weighing from 3000 to 4000 lbs. with a correspondingly low fall, although the energy of the blow is the same in both cases; the blow with the high fall being largely taken up in bending and brooming the pile, while that with the low fall seems to coax along the pile, as it were. Whether this results from the fact that more blows can be made in the same interval of time, thereby keeping the pile in constant motion, rather than allowing intervals of rests, or from some other cause, the fact is indisputable, and greater depths can be reached with less damage to the pile.

4. In driving piles in certain kinds of clay, the lateral spring of the pile makes a hole perceptibly larger than the pile itself,

thereby allowing surface water to percolate along the pile, often as deep as the point of the pile; and whether it ever gripes the pile is an unsettled question. This, combined with vibrations from a rapidly moving train, may ultimately cause settling.

5. No rule that does not take into consideration the loss of energy resulting from broomed ends, the varying amounts of frictional resistance of different materials during the process of driving, and the depth of the pile in the soil, but is based solely on the weight of the hammer, the height of the fall, and the penetration at the last blow, can furnish any reliable or even approximate idea of either the immediate or ultimate supporting power of a pile or any number of piles.

84. We must therefore rely mainly on experience, or upon experiment, in each particular case; and in the absence of these, it is merely guess-work and taking the chances. Experiment, however, is in the reach of all, will cost but little money, and will take but little time; and no excuse can be given for not making satisfactory tests of some kind in the absence of precedents in similar material or in the same locality. If experimenting on piles in trestle work, drive a single bent or two bents of piles at the proper distance apart. Upon these construct a platform, and place weights equal to or twice as great as the greatest load that can possibly come upon them. If under this load no settlement takes place, the trestle will be safe, and piles in other bents driven to the same depth and to the same resistance in the last few blows can be relied upon. If, on the contrary, settlement does take place, more piles or longer piles must be used. Weights can gradually be added on a single pile until it begins to settle, and from this the number of piles can be estimated to carry any proposed load, allowing a factor of safety from 2 to 4. Clusters of piles will however bear more in proportion than single piles, if not driven nearer thar $2\frac{1}{2}$ ft. centres, as they consolidate and compact the soil in proportion as the numbers increase in a given area. Such experiments should not be made for at least 24 hours after the piles are driven, so as to allow time for the material

to compact and be adjusted around the pile. The following is an interesting and instructive experiment made by Maj. E. T. D. Myers. I give it substantially in his own words, as contained in a letter dated Feb. 7, 1885:

"Bents 12½ ft. centres, 6 piles each; length, 50 ft.; Grade line, 15 ft. above low-water; in use fourteen years. Piles driven in a liquid mud. Two bents of 6 piles each were driven, upon which a platform was placed, and upon this a weight of 75,000 lbs. uniformly distributed. The experiment was made 19 hours after driving.

O	O	O	O	O	O	Bent 18th.
1	2	3	4	5	6	
O	O	O	O	O	O	Bent 17th.
1	2	3	4	5	6	

No settlement taking place, piles Nos. 2 and 5 in each bent were cut out, leaving 4 piles in each bent. Then No. 3 of the 17th and No. 4 of the 18th bent were cut out, leaving only 3 piles in each bent. About 5000 lbs. was then added to the load, when No. 6 of 18th bent yielded, followed by No. 3 of the same bent, and sank until Nos. 4 and 5 were again brought to bear. It required, therefore, about 13,000 lbs. each to start the piles. The record of the driving was as follows:

BENT 17: FALL FROM 3 TO 10 FT.

Pile	No.	1,	11 blows.	Last	blow,	7	ft. fall,	drove it	11	inches.
"	"	2,	13 "	"	"	9	" "	"	9	"
"	"	3,	8 "	"	"	9	" "	"	18	"
"	"	4,	8 "	"	"	8	" "	"	17	"
"	"	5,	9 "	"	"	5	" "	"	6	"
"	"	6,	7 "	"	"	7	" "	"	10½	"

BENT 18.

Pile	No.	1,	12 blows.	Last	blow,	5	ft. fall,	drove it	10½	inches.
"	"	2,	8 "	"	"	4	" "	"	8	"
"	"	3,	8 "	"	"	4	" "	"	8½	"
"	"	4,	9 "	"	"	3	" "	"	4	"
"	"	5,	14 "	"	"	10	" "	"	9	"
"	"	6,	5 "	"	"	9.8	" "	"	22	"

A pile 40 ft. long, after sinking 30 ft. with its own weight and that of the hammer weighing 2000 lbs., was struck with a blow of 2-ft. fall, and then settled 6½ ins. in one minute by the weight of the hammer. Four weeks after this a blow with a fall of 5 ft. did not move it. A blow of 14-ft. fall drove it 4½ in. Also at the Gunpowder River piles 40 to 50 ft. long were driven, until they did not sink more than 18 in. under a hammer weighing 1800 lbs. falling 20 ft. Four piles to the bent. In neither case was a hard stratum passed through or reached." This is but the common experience in the Southern swamps. Even with very light falls, the penetration at the last blow is from 4 ins. to 2 ft. High falls are out of the question, as there is danger of losing both pile and hammer. In all cases above alluded to, these trestles have carried without settling the heavy trains of the present day. The above examples show the great load that piles driven to a depth of 30 to 35 ft. in the softest material that can be called earth will bear. A 30-ft. pile that is 30 ft. in the soil would present on an average about 90 sq. ft. of surface in contact with the soil, and bearing safely 13,000 lbs.; the frictional resistance would be about 144 lbs. to the sq. ft. of surface. The probabilities are that they would carry to at least 300 lbs. The frictional resistance is known to vary from 300 to 800 lbs. per sq. ft., depending upon the nature of the material into which the piles are driven. It will be observed in the above table that pile No. 6 of the 18th bent was the first to yield under the weight of 13,000 lbs. (This pile penetrated under a 9.8-ft. fall at the last blow 22 ins.) The effect of this was to throw a large portion of the 13,000 lbs. on the next pile in the same bent, which of course yielded. The greatest load that could come upon a span of 12½ ft. would be 75,000 lbs.; or, in a four-pile bent, would be 18,750 lbs., and in a six-pile bent 12,500 lbs. per pile.

85. Some 8 miles of trestle, constructed under the writer's direct supervision in the Southern swamps, the bents containing 4 piles, spans 12½ ft., depth of pile in the soil varying from 30 to 35 ft., the penetration varying from 6 in. to 2 ft. at the last blow of a 2000-lb. hammer falling only a few feet.

has carried for twenty years the heaviest trains without any settling. In the abutments of some of the bridges in these swamps the piles have carried with perfect safety 17,000 lbs. to the pile. How much more they are capable of carrying is not known. In one of these abutments, piles only 30 ft. in the soil could not be moved by continued hammering with high falls a few days after driving. The experiment was made as the writer was not satisfied with the record of the original driving, and desired the piles to be driven to a greater depth. Finding it impracticable to move the piles he determined to hammer one or two to destruction or move the piles; destruction was the result, and new piles were driven to take their place.

86. We may, therefore, conclude that piles, from 30 to 40, in even the softest alluvial soils, will carry, by frictional resistance alone, from 20,000 to 25,000 lbs., or 10 to $12\frac{1}{2}$ tons. There are examples of piles driven in stiff clay to the depth of 20 ft., that carry from 70 to 80 tons per pile; this is an unnecessarily heavy load, and when driven from $2\frac{1}{2}$ to 3 ft. centres they will rarely have as much as one-half of the above loads to carry. There are many instances in which piles carry from 20 to 40 tons under the above conditions.

87. In sand and gravel, piles will carry to the full extent of the crushing strength of the timber, provided the depth in the material is sufficiently great to prevent vibrations from reaching the point of the pile; other considerations will require this depth to be at least 10 ft. or, at most, 20 ft. Any further hammering on piles in such materials is a waste of time and money, and injurious to the pile itself. To hit such a pile 100 to 150 blows to drive it an inch, as has been done, is simply folly.

88. Some times piles drive easily and regularly to a certain depth, and then refuse to penetrate farther; this may be caused by a thin stratum of some hard material, such as cemented gravel and sand or a compact marl. It may require many hard and heavy blows to drive through this, thereby injuring the piles, and perhaps getting into a quicksand or other soft material, when the pile will drive easily again. If the depth of the overlying soil penetrated is sufficient to give lateral stability, or if this can be secured by artificial means, such as throwing in broken stone or gravel, it would seem unwise to endeavor to penetrate the hard stratum, and the driving should be stopped after a practical refusal to go with 2 or 3 blows. The thickness of this stratum and nature of the underlying material should be either determined by boring or by driving a test pile to destruction if necessary. In the latter case the driving of the remaining piles should cease as soon as the hard stratum is reached.

89. Sometimes in driving piles it is difficult to keep the piles down after the impact of the blow is over: the piles, beginning to rise, lifting the hammer with it, and upon removing the hammer the piles would shoot up 5 to 6 ft. or more. This is, no doubt, due to a stratum of quicksand. The writer has overcome this difficulty by driving the piles with the butt, or large end, downward. This is the only case in which piles were driven butt downward, in the writer's experience, though some authorities recommend it.

90. The above seems to cover the various conditions and kinds of material met with in driving piles, and, as can be readily seen, no general or rigid rule can be given, either as to the depth to which a pile should be driven into any kind of soil, or as to the penetration in the last blow, or last few blows. Experience, and experiment alone can be of any practical value. But enough has been said to establish, first, that when piles are driven in a soft, swampy material, a penetration of from 30 to 40 ft. into the soil will give ample support for ordinary purposes, regardless of the weight of the hammer, the height of the fall, or the penetration at the last blow, within limits generally existing in practice; second, in clay, sand, and gravel the depth required is only that necessary to give stability to the structure, to get below the scour-line, or beyond the reach of vibrations caused by moving loads, in general, from 10 to 20 feet; third, that a continued hammering on piles, after practical refusal to go, is absolutely injurious.

91. The following formulæ are given for the benefit of those

who may differ with the opinions expressed above. Rankine gives the following: The energy of the blow is employed as follows: $Wh = \frac{P^2 l}{4ES}$ (employed in compressing the pile) $+ Px$ (employed in driving it), in which $W =$ weight of the ram or hammer, $h =$ height of fall, $x =$ the penetration of the pile at the last blow, $P =$ greatest load that it will bear, $S =$ area of cross-section of the pile, $l =$ length of pile, and the modulus of elasticity $E =$ about 108,000 lbs. per square feet. Hence

$$P = \sqrt{\frac{4ESWh}{l} + \frac{4E^2S^2x^2}{l^2}} - \frac{2ESx}{l} \quad . \quad . \quad . \quad (1)$$

In any particular case the values of the quantities in the second member are all known, and P can be found. Major Sander's formula is as follows:

$$P = \frac{h}{a} \times \frac{W}{8}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

in which $h =$ fall in inches, $W =$ weight of hammer in lbs., $a =$ penetration at each blow towards the last, $P =$ safe load in pounds. Trautwine's formulæ,

$$P = \frac{\sqrt[3]{hw} \times 0.0268}{1 + a}, \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

$$P = \frac{50 Wh}{a + 1}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

and for the safe load take one half of this value. Assume the weight of the hammer at 2500 lbs., penetration $1\frac{1}{2}$ inches at the last blow, or towards the last. Then from eq. (2) the safe load

$$P = \frac{40 \times 12 \times 2500}{1.5 \times 8} = 100{,}000 \text{ lbs.} = 50 \text{ tons,}$$

and from eq. (3) the safe load equals

$$\frac{P}{2} = \frac{3.42 \times 2500 \times 0.0268}{2.5 \times 2} = 49\tfrac{1}{2} \text{ tons, or } 99{,}000 \text{ lbs.,}$$

the height of the fall being taken at 40 ft. in both cases. The calculation in Rankine's formula is long and tedious, and probably no more accurate. Applied to piles driven with a hammer

1200 lbs. and fall of 20 ft. penetration $\frac{3}{4}$ in. Trautwine's formula gives, as a safe load, 24.9 tons, and Major Sander's 21.4 as safe load; the actual load borne by the piles is 18 tons to each pile; and again, piles driven only 16 ft. into alluvial mud, weight of hammer 1500 lbs., fall 24 ft., penetration 2 in., actually supporting 20 tons. By Trautwine's formula safe load is 19.3 tons, and by Major Sander's 12.06 tons, and still in another case the calculated safe load is 55 tons, whereas the actual load is 70 tons. In New Orleans the piles driven from 25 to 40 ft. carry safely from 15 to 25 tons. This is in a soft, alluvial soil.

Article XLIII.

TIMBER PILES—(CONTINUED).

92. There has been suggested recently another formula, known as the *Engineering News* formula, as follows: $P = \frac{fwh}{S + C}$, $P =$ safe-bearing resistance, f a factor varying from 12 to 1, and recommended to be taken $= 2$, giving a factor of safety of 6, $w =$ weight of hammer in lbs., $S =$ penetration in inches, the average during the last few blows, and C taken $= 1$, a constant to provide for the increased resistance to moving at the moment of impact, reducing the formula to $P = \frac{2wh}{S + 1}$ for practical use. Since writing the above pages on pile-driving, this formula has been brought into great prominence by reason of the learned and able discussions, as to its theoretical accuracy and practical reliability and usefulness, by some of our leading engineers. The conclusion seems to be reached that it is certainly as reliable as any of its predecessors, and perhaps comes as near being reliable as it is practicable, though leaving out many important conditions and considerations, which must materially modify the relations between the energy of the blow and the penetration. The formula is simple and easy of application in any particular case. The writer, however, sees no reason to modify the already expressed opinion that the ultimate bearing resistance of

piles cannot be expressed even approximately in terms of the weight, fall, and penetration; and even if approximately true for one kind of material and one set of conditions usually attending the driving, they would miss by very far the mark when applied in case of another material and under other conditions. And especially as his experience has been confirmed during these discussions by the statements of many prominent engineers as to penetration at the last blow, one engineer stating that piles about 40 ft. long sunk with their own weight and that of the hammer the full length of the pile, and could not be driven at all after a period of rest; that they have carried ever since the heavy trains used on the road.

The writer has sunk piles from 6 to 10 ft. simply by working the piles backward and forward; two such piles to the bent carried safely a construction train, loaded with rails and ties, for many months without any evidence of settling.

93. After a period of rest it is evident that piles support their loads by the upward pressure at the point of the pile and by the frictional resistance on the surface of the pile in contact with the soil. The relation between these resistances and the weight that the pile can carry can be simply expressed as follows: $w = p + fs$, in which w = the safe-bearing power, p = the safe resistance to settling determined by the bearing power of the material, f a factor depending upon the frictional resistance of the material on the surface of the pile, s = number of square feet of surface in contact with the soil.* If we knew p and f in all cases, and the load to be carried, we could determine the depth of one or a group of piles below the surface necessary to carry the load. The value of p is already known approximately for ordinary materials, and for sand, gravel, and clay is universally recognized as safe at 5000 to 6000 pounds per sq. ft., and for silt can be taken at zero. The value of f can be determined with the same degree of accuracy as is now used

* There is no other formula applicable to the bearing power of piles sunk by the water jet, or worked into the ground by a to-and-fro motion, or when driven 30 or 40 ft. into the soil by three or four blows with a hammer falling three or four feet.

and considered safe in the usual coefficients of friction, and at a comparatively small cost; and, in the absence of more reliable information, it could be taken at from 100 lbs. in the softest semi-fluid soils to 200 lbs. per square foot in compact silt and clay, and from 300 to 500 lbs. in mixed earths with considerable grit, and from 400 to 600 lbs. in compact sand, and sand and gravel. Assuming, then, that we were driving piles for a trestle, 4 piles to the bent, bents 14 ft. apart, and assuming the equivalent uniform load to be 6000 lbs. per foot, each pile would have to carry 21,000 lbs.

In the silt of the swamps, with $p = 0$ and $f = 150$ lbs. the formula gives: $s = \frac{w - p}{f} = 140$ sq. ft. of surface, a pile averaging 11 ins. diameter contains 2.8 sq. ft. per foot of length, and should therefore be 50 ft. in the ground. The writer is satisfied that a bent of four such piles, especially if the outside piles batter, would safely carry the load. If considered risky, put in a centre pile, reducing the load per pile to 16,800 lbs.

2. When driven in clay, $p = 5000$ lbs., and $f = 150$ lbs., each of the four piles would have only to carry $fs = w - p = 21{,}000 - 5000 = 16{,}000$ lbs. by frictional resistance, hence $s = 106$ sq. ft., or depth in the ground $= 38$ ft., and if $f = 200$ lbs., $s = 80$ sq. ft., and the depth in the ground 30 ft. No one would question that ample safety is secured in this case, and in fact 15 to 20 ft. in the ground would be perfectly safe.

3. In compact sand, $p = 5000$, $f = 500$, $s = 32$ sq. ft. and depth in the ground $= 12$ ft. nearly. This would answer in any case, unless danger from scour exists. On any reasonable values of p and f, the above formula, I think, would certainly be equally as reliable as any other, and certainly comports better with the actual existing conditions, and with a fair number of practical tests similar to those already described, in varying soils, would give us as fair a standard of comparison, as now exists in the case of retaining walls, timber, and iron columns and beams, which are based upon experimentally determined constants.

93½. We can, then, conclude that the bearing power of piles will vary from 13,000 lbs., or 6½ tons, to 140,000 lbs., or 70 tons, according to the character of the material into which they are driven; and this, reduced to frictional resistance per sq. ft., for a pile driven 30 ft. into the soil, gives from 140 lbs. to 1550 lbs., which may be taken as the extreme limits, and from 200 to 800 lbs. may be taken as good working limits, not to exceed the smaller, in alluvial and soft soils, nor the greater, in the firmer materials, such as stiff clay, sand, and gravel or mixed materials.

94. The usual mode of driving piles is by means of the pile-driver, which consists essentially of 2 horizontal pieces of timber, 10×12 ins. or 12×12 ins., and 10 to 18 ft. long, placed about 3.5 ft. apart, connected by short struts and tie rods; near one end of these, two uprights, 6 or 8 ins. by 10 ins. and from 20 to 40 or more feet long, are connected by cutting shoulders in the ends, so as to fit the horizontal pieces, to which they are also fastened by bolts. In the rear of the verticals bent iron bars are placed, the ends passing through the uprights or leads; these act as braces for the leads, and also to hold the wedges necessary to keep the piles in place and straight. These are placed at intervals of from 6 to 8 ft., vertically; on top of the leads a strong cap of oak or some hard wood 6 ins. thick and 12 to 15 ins. broad, is placed and connected by mortise and tenon and iron bolts. A ladder runs from the other end of the horizontal frame nearly to the top of the leads to which it is bolted; at intervals horizontal pieces connect the ladder and the leads, upon which planks are placed for platforms. On the inside of the leads a strip of hard wood 2½ to 3 ins. square is bolted at close intervals, and on the face of these-strap iron ½ in. thick is bolted, the heads of the bolts countersunk. A cast-iron hammer of the required weight, varying from 1000 to 4000 lbs., is cast with grooves on the sides, so as to be held in place by the strips, and at the same time to slide freely on them. A rope is attached to the upper end of the casting, and passes through a hole bored in the cap. and over a pulley fastened on top of the cap, thence downward,

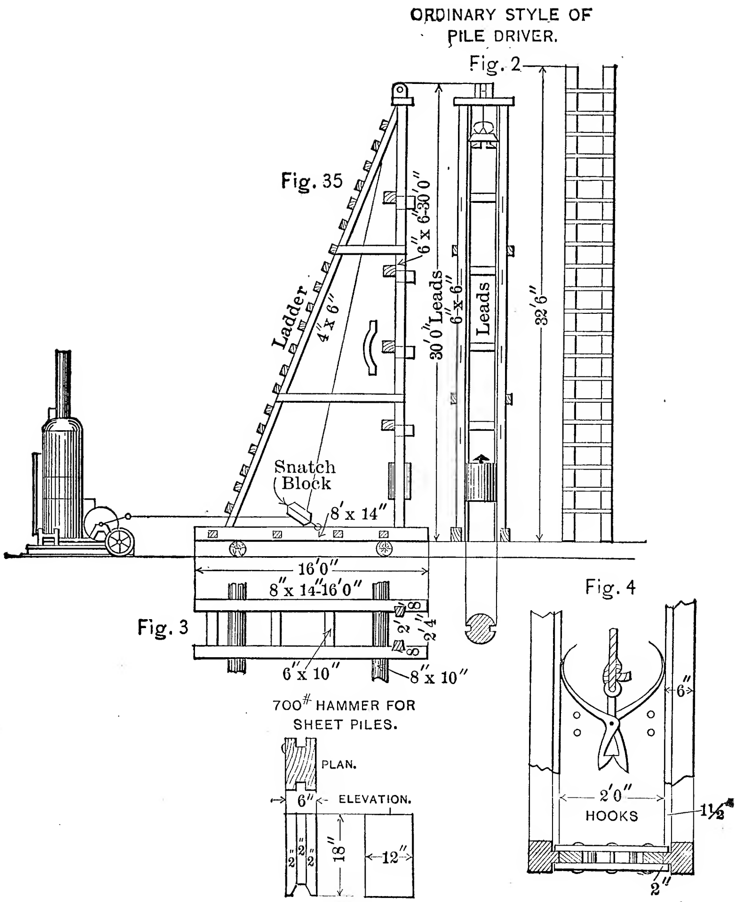


FIG. 35.

[*To face page* 223.]

passing through a snatch block at the bottom, and thence horizontally to a drum or capstan; this is now the ordinary arrangement, the rope being permanently fastened to the hammer. The second plan is to attach the rope to a heavy double block of wood, into which is framed a pair of nippers, with the upper ends curved outward, and the lower ends with pyramidal points and square projecting shoulders on the inside; these are so suspended on a strong bolt that the lower ends remain in contact, and are only opened by closing the curved upper ends. The top of the hammer has a wedge-shaped projection with square shoulders a few inches from the top of the projection. The block is also framed so as to slide down the strips on the leads by its own weight, when, in falling rapidly on the hammers, it takes hold of the projection, when the power is applied it lifts the hammer with it, when it comes in contact with bevelled blocks fastened to the leads near the top, the curved upper ends are gradually closed, the lower ends open, and the hammer falls; the block again descends rapidly and clutches the hammer as before. The leads are braced laterally, by inclined struts resting against horizontal pieces projecting on either side, and the rear end of the horizontal frame must be weighted or held down, so as to counterbalance the weight of the hammer. The first method, in which the rope is attached directly to the hammer, has many advantages; more blows can be struck per minute, the height of fall can be more easily regulated and changed, being dropped at any desired distance above the top of the pile, and there is no danger of losing the hammer if the pile should spring out of the leads or be driven below them. (See Fig. 35, (1), (2), (3), and (4).)

95. Such is the simple pile-driver. On firm ground it can be moved from point to point by letting it rest on hardwood rollers attached to the under side of the horizontal frame, these rollers being turned by levers inserted in holes bored into them. On softer ground a platform of timbers can be laid, on which the driver rests and is moved. This is a slow method, and where any great distance is to be passed over it is best to fasten the driver to a platform made of strong timbers, upon

which also the engine and boiler can be placed, the whole then resting on strong timbers, to the under side of which iron bars or rails are fastened. This is elevated on a level with the top of the piles, the leads project beyond the platform a distance equal to the distance between the bents, say $12\frac{1}{2}$ feet; the two centre piles of the bent can then be driven. The driver is moved forward a few inches, the frame holding the leads can be turned on a pivot, and the two outside piles driven in a proper line with the inside ones. These piles are then cut off and capped, temporary stringers placed in position, iron castings with grooved rollers fastened to the stringers, upon which the rails run, and by ropes attached to the bent and the drum of the engine the driver is pulled forward into its new position, and another bent driven as before described, and so on. In a more perfect form the driver is attached permanently to a platform or railway car, and as the work proceeds stringers and rails are temporarily laid, and the car run forward on these. For the repairs of completed roads, drivers of this kind are used, the leads being hinged so that when not in use they can be lowered, so as to pass through bridges, tunnels, etc. For driving piles in water the driver is simply fastened to a barge, and floated to its position, controlled, held, and moved for short distances by means of anchors. Piles can be handled more readily and more economically on water than on land, but it is more difficult to place and hold the driver when floating, especially if the current is rapid, or in high winds. The cost of driving piles should not exceed 8 or ten cents per lineal foot of pile. The cost of piles vary from 9 to 12 cents per lineal foot.

96. The power used in pile-driving is either man, horse, or steam power. The first is not often used. It is necessary to have a light hammer and a low fall. A number of men take hold of a rope, lift the hammer a few feet, and then all let go at the same time; it is a slow process, and not calculated to obtain the best results.

Horse power is very common on land, and can be used on water; the rope is fastened to a capstan, which can be readily made by any carpenter; a long lever is attached to a centre post, to which a horse is attached, and as the horse moves the rope is wound around the capstan, and the hammer is lifted; at the proper moment the capstan can be thrown out of gear, and the hammer falls. Good and rapid work can be done in this manner.

But when a large number of piles are to be driven, steam power is mainly used; the rope is attached to an iron spool connected with the engine, around which the rope is wound as the power is applied, and by throwing it out of gear the hammer falls. This is the most rapid and expeditious method, and admits of very heavy hammers being used.

There is also a steam-hammer pile-driver, in which the blow is struck by a hammer attached directly to the piston of an engine. In this very powerful and rapid blows can be struck, and doubtless it has many advantages; but it is not in common use, and in fact it is seldom seen, and therefore it can be presumed to be less economical than the ordinary drivers.

97. None of the drivers above mentioned can be used to drive piles inclined to a vertical without great inconvenience and delay, but it is often desirable to drive piles on a batter, this method possesses a great many advantages in driving piles for trestles, as will be shown in another paragraph. A pile-driver is constructed for this purpose somewhat differently from those described above. Instead of the leads being fastened to the horizontal frame, they are supported by strong heavy bolts, attached to an iron frame, which is fastened to the horizontal frame, the leads being free to turn about the iron bolt through an arc of many degrees, by which means the leads are inclined to the vertical, and the pile can be driven in the desired direction with the same rapidity as in other cases. The construction of such a driver is as simple as those used in driving only vertical piles, and should be used more generally than it is. The drawings, Fig. 35, (1), (2), (3), and (4), show the general construction of pile-drivers.

98. When for any reason it is necessary to sink piles to a great depth in a firm and compact material, without injury to the piles by many heavy blows, it can be done by the use of the water-jet; a pipe can be attached to the side of the pile either fitting in a groove cut for the purpose or fastened to the outside; this pipe ending in a nozzle at the point of the pile, the upper end attached to a force pump by a hose. When the water is forced through this pipe it removes or loosens the material around and under the point of the pile, which sinks by its own weight, or a weight is placed on top, or aided by light blows from a hammer. This is a rapid and effective mode of sinking piles, and has been used to a large extent and with satisfactory results. It should be infinitely preferred to the practice of long-continued and heavy blows to drive a pile a few inches. Piles have been sunk to great depths by this process; it is an application of the same principle as has been fully explained in sinking the Cushing cylinders and in making borings for foundations. An oblique hole is sometimes bored into the pile near the bottom, so as to discharge the water exactly at the point of the pile; but this does not seem to be necessary, or even to possess any material advantage; a few blows should be given after stopping the water-jet. It is of great advantage in sinking iron pipes to rock for the purpose of submarine blasting, or for columns composed of iron cylinders, as well as in sinking screw-piles, which will be explained under that heading.*

ARTICLE XLIV.

USES OF PILES.

99. PURPOSES for which piles are used will now be discussed. Piles are divided into long and short piles, or piles to bear directly and entirely the load, and piles the main object of which is to compact a soft and loose material so as to increase the bearing power of the soil. Short piles or those used for the latter purpose are from 8 to 15 ft. long, and generally from 8 to 10 inches in diameter; these are used principally to sup-

* Piles were recently sunk with the water-jet to the depth of 25 ft. in sand. It is stated that it only required two minutes to sink each pile. Bowlders under the points of piles were carried down with the piles by sinking a pipe to the under side of the bowlders and using two water-jets at the same time. These piles could not be moved by blows from a heavy hammer only a few minutes after stopping the flow of water from the pumps.

port the walls of houses and other comparatively light structures. In some sections of the country, especially in the Southern cities, the soil is of a soft alluvial material, and in its natural state is not capable of bearing heavy loads. In such cases trenches are dug, as in firmer material, and a single or double row of short piles are driven close together, and under towers or other unusually heavy portions of the structure the area thus to be covered is filled with these piles; the effect of this is to compress and compact the soil between the piles and to a certain extent around and on the outside, thereby increasing its bearing power, whatever resistance the piles may offer to further settlement may be added, though not relied upon. These piles are then cut off close to the bottom of the trench, and generally a plank flooring is laid resting on the soil and piles, or a layer of sand or concrete is spread over the bottom of the trench to the depth of 6 ins. or 1 ft., and the structure whether of brick or stone commenced on this. There is little or no danger of such structures settling, and if they do the chances are that they will settle uniformly if the number of piles are properly proportioned to the weight directly above; but if the same number of piles are used at all points of the structure, although considerably great weights are on some walls or some parts of a wall, unequal settlement may take place, causing ugly or dangerous cracks in the structure.

100. A modification of this plan is to drive a pile into the soil and then withdraw the pile and fill the hole thus formed with sand; this being done at intervals of 2 or 3 ft. under the walls of the structure as above described and all of the holes filled with sand, there results a good foundation. The columns of sand are called sand-piles, owing to the great mobility of the sand grains; they act somewhat as in a fluid pressure, pressing equally in all directions at any given depth, and therefore afford a better support than the wooden piles, and have the further advantage of being permanent. The wooden piles, unless constantly wet, will rot sooner or later, and although timber constantly wet does not rot, yet it becomes more or less softened and soppy and loses some of its original strength.

Short piles are also used under small piers and abutments for the same purpose as above mentioned, but should not be used where very heavy weights are to be carried, or where they can be reached by vibrations caused by rapidly moving trains; in such cases long piles should be used.

101. Long piles vary in length from 15 to 100 ft. and in diameter from 12 to 18 ins. or more; and although their resistance to settling is increased by compacting the soil, the load is supported generally, though not always, directly by the pile, and the weight does not come upon the soil between them or around them at all. Such piles are always used when great depths are to be reached below the ground or water-surface, or where there is any danger of the material around them scouring out—as under very large and heavy piers and abutments in rapid currents, or in very soft materials; also, where piers or abutments are built on or near to the edge of steep banks which are in danger of caving in, under the action of rising and falling water, although the material itself may have ample supporting power. This is the case along many rivers, notably the Ohio, whose banks cave in regularly and annually at very many points, unless protected by stone-paving or vegetable growth, willow trees, etc. Should such banks cave under the piers, broken stone could be thrown in and around the piles, thereby securing the structure. They are also used in constructing wharves, dykes, etc., for the main piles in coffer dams, and to a very large extent in building railroads across swamps, bayous, sloughs, etc. In driving piles under piers and abutments, the piles are driven in rows about $2\frac{1}{2}$ feet from centre to centre of piles in all directions, over the entire area to be occupied by the structure and one row on the outside, making the area about $1\frac{1}{2}$ to 2 feet larger all around than the bottom of the structure itself. After driving the piles they can be cut off at or above the bottom of the excavation, and upon these caps of 12×12 ins. timber are placed and drift-bolted, another layer of timber placed at right angles, and on top of this another layer of timber or plank, all bolted or spiked together, and the masonry started on this. All of the timber should be under the lowest low-water, so as to be constantly wet; in this case the piles bear the entire load. Sometimes concrete, broken stone, or gravel is placed around the heads of the piles and around and between the timbers of the platform, and again the timber may be entirely omitted and concrete placed around and over the piles to a depth of at least 2 feet. In the last two cases a part of the load is borne by the compacted soil between and around the piles. Opinions differ as to which is the better: both are good enough; but unless the timber is wet it would seem better to leave it out, though some authorities say that timber imbedded in cement concrete will last as long as when constantly immersed in water. If so, the first, or rather the combined timber and concrete, would be preferred. Both methods are in common use.

102. In the construction of wharves, piles are driven in rows extending well out into the water; the distance apart of the piles and the depth to which they must be driven depending entirely on the load which they have to bear. These are cut off a few feet above the water surface, capped with square timber 10 × 12 ins. or 12 × 12 ins., upon which joists are placed at close intervals, and on these a plank flooring of hard wood 3 ins. thick. Each pile in such cases supports an area whose sides are respectively the distance between the rows and between the piles in each row—a fair average being 5 ft. each way, or an area of 25 sq. ft., as very heavy loads are often concentrated over small areas. The proper area is, however, easily determined when the greatest load per square foot is decided upon. These piles will soon rot above the water-line, when they must be cut off, and framed bents of timber constructed on them. The spaces are, however, often filled up with earth, gravel, or shell, and only a timber wall or bulkhead must be maintained at the outer end to hold the material in place. This consists of a timber crib resting on two or three rows of piles, tied back into the embankment by long timbers notched and bolted to the crib timbers. In front fender piles are driven and fastened to the crib by iron straps or bolts, and projecting above the wharf 4 or 5 ft. The bulkhead should be strong and heavy and well anchored to the earth as the weight of the material behind and the load upon it exerts a great force, tending to force it outward; it would be better to drive the piles under the crib, slightly inclined backward, so that the resultant pressure would be nearly in the direction of their length. On important water-fronts, masonry or concrete walls or bulkheads are constructed and faced with timber, against which boats or vessels can rest. Bulkheads are subjected to severe blows and shocks from vessels. Such constructions are used along the foot of embankments, to prevent caving and sliding.

103. Dykes or jetties are formed by double rows of piles, driven at right or acute angles with the direction of the current, sheeted as in case of walls of coffer-dams, and filled with gravel, stone, or shells. When driven at a slight inclination to the current they serve to force the water through a narrow channel, and the increased velocity scours the material of the bed of the river, depositing it behind the jetties or in the deeper water below the mouth of the jetties. When at a large or a right angle to the current they serve the same purposes. They should not be built too high above the low-water surface. These subjects, however, more properly belong to river and harbor improvements, and are merely alluded to incidentally.

104. The more extensive use of piles for foundations is in the construction of trestles across swamps, bayous, etc. There are two methods practised; one is to drive piles in rows of 4 or 6 each, the rows being from 10 to 25 ft. apart, the position of the piles in each row or bent being regulated so as to be under the posts of the structure above. They are then cut off a little below low-water or the moisture surface, and upon these framed trestles of any height and of any of the forms above described are constructed and fastened to the piles by straps or bolts. This has the advantage of placing the piles where they will do the most good, and of utilizing the full length of pile to support the load by direct bearing or by the frictional resistance of the soil. In the second case the piles are allowed to project above the ground or water and then cut off; the caps resting directly upon the tops of the piles, to which they are fastened by bolts or straps. This method is hardly applicable when the height of the trestle is more than 20 to 25 ft. above the ground or bed of the streams, owing to the great length of piles required, and the further fact that the piles are driven vertically, and consequently would not have breadth of base sufficient to provide lateral stability, and it is therefore limited to heights of trestle rarely exceeding 15 ft. above the ground. This is generally as great a height as will be required above the swamps, and commonly not more than 5 to 10 ft. When no batter-posts are used, the common construction in a 4-pile bent is to drive the two inside piles about 5 ft. centres, so that they may be directly under the rails, and the outside piles $2\frac{1}{2}$ ft. from these on either side; making the total distance from outside to outside of piles from 12 to 13 ft. This requires caps about 14 to 15 ft. long, generally fastened to the piles by drift-bolts. Upon these stringers, cross-ties, and guard-rails are placed similarly in every respect to those of framed trestles. But often additional stringers are placed over the outside piles, and long cross-ties 13 to 14 ft. long are used. This requires a considerable increase in the quantity of timber and iron, with hardly any compensating advantages, as seen by the following comparative estimate. For a single span, $12\frac{1}{2}$ ft., we have

1 cap 12 × 12 in. × 14 ft.	168	ft. B. M.	
4 stringers 6 × 14 in. $12\frac{1}{2}$ ft.	350	"	
12 ties 6 × 8 in. × 9 ft.	432	"	
2 guard-rails 6 × 8 × $12\frac{1}{2}$ ft.	100	"	
			1050.00 B. M.

And in the second case

1 cap 12 × 12 in. × 14 ft.	168	
6 stringers 6 × 14 in. $12\frac{1}{2}$ ft.	525	
12 ties 6 × 8 × 14 ft.	672	
2 guard-rails 6 × 8 × $12\frac{1}{2}$ ft.	100	
		1465.00
Or an excess of timber for each $12\frac{1}{2}$ ft. length,		415.00 B. M.,

equivalent to a waste of 175,300 ft. B. M. per mile of trestle, at \$26 per 1000 ft. B. M. = \$4558 per mile. Practically but little increase of strength is gained. There is less danger of a train wrecking the trestle or of tumbling off, which of course is an important consideration. Whether the additional safety secured is worth the expenditure of the above sum per mile might be a question; but this can hardly be justified when framed trestles on the same road do not generally provide for it, where it would seem that every precaution for safety should be provided, as the trestles are much higher, and consequently the destruction to life and property would be much greater should a train leave the track and be thrown from the trestle; if considerations of safety control in one case they should control in all. It is evident that, in a 4-pile bent, with all piles vertical, the two piles under the rails have nearly if not all of the weight to carry, the outside piles only acting when the middle ones settle. For this reason some engineers use the 3-pile bents, the inner piles placed half-way between the rails; the outer piles about $1\frac{1}{4}$ ft. from the rails on the outside. These are capped as usual, and the stringers placed between the piles and only one half the distance from the outside pile that they are from the middle pile. By this arrangement each outside pile bears two thirds of the weight on one rail, and the middle pile one third of the weight from each rail, or two thirds in all. All of the piles will bear an equal portion of the load. But the breadth of base will be reduced to $9\frac{1}{2}$ to 10 ft. from outside to outside of pile, the bent, therefore, will be wanting in lateral stability. Moreover, as the weight is supported by the caps, at a point between the piles the caps will have a bending strain to bear, and though not requiring caps of greater dimensions than commonly used they will require a more frequent renewal to insure perfect safety. In this trestle the only saving is one pile for each bent, or each $12\frac{1}{2}$ ft., or 424 piles to the mile; assuming the average length of pile at 40 ft. we save 16,960 lineal ft. of piling at 30 cents a ft. = $5088. For low trestles there will be sufficient strength and stability. The caps are 12 ft. long, so there is saved also about 10,000 ft. B. M., to which may be added for saving in the length of the diagonal or X bracing some 5000 ft. B. M., a total of 15,000 ft. at $26=$390, or a total saving of $5478 per mile. Some roads take advantage of this saving

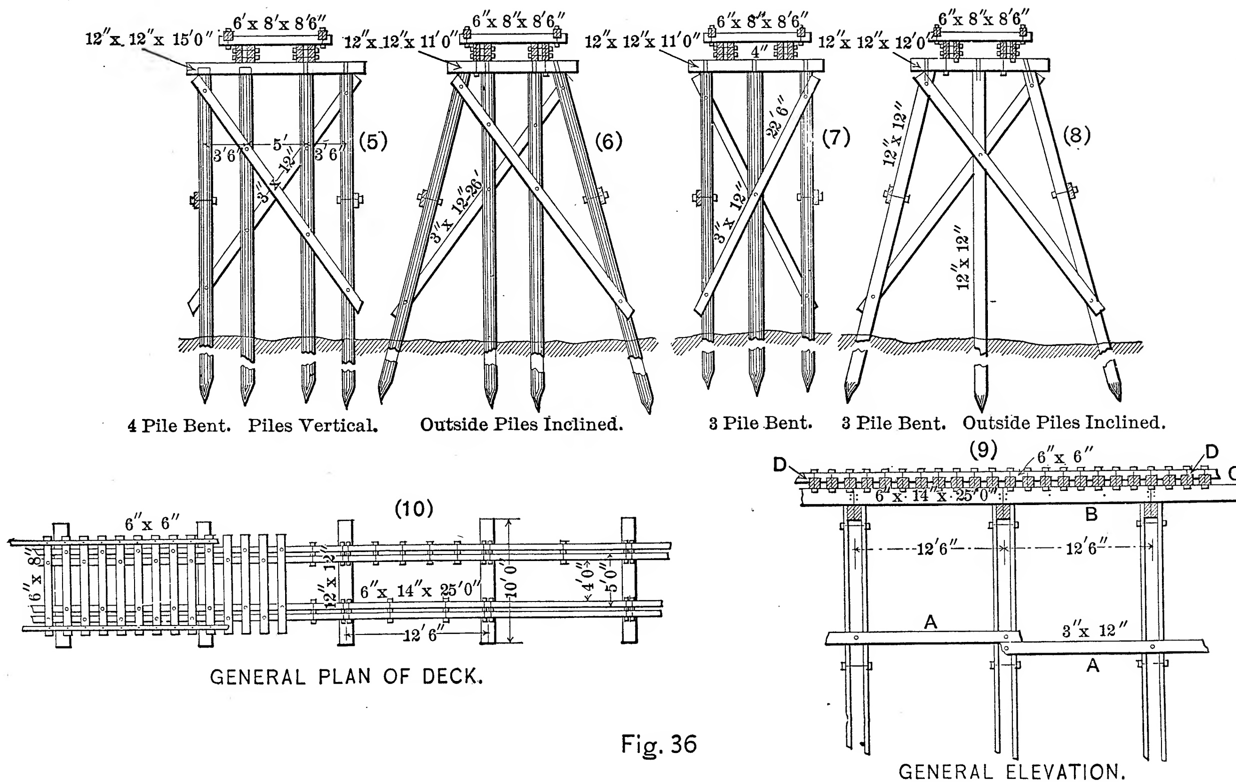


Fig. 36

[*To face page* 233.]

and use the 3-pile bents, but the 4-pile bents are preferred and are more commonly used.

If we are to use the 4 pile bent the piles should be driven so that they may all do full service not only in bearing the load, but in adding stiffness and strength to the structure; or, in other words, all trestles should be built in such a manner as to make the most advantageous use of the parts composing them. This can be done by driving the outside piles on a batter of 3 ins. per vertical foot, the same as that used for the batter posts in framed trestles (the driver for such piles has already been alluded to). It costs no more than the 4-pile bent with all vertical piles; is as easily and as rapidly built. All of the piles carry a portion of the load; the outside piles give stability and stiffness to the structure, when it becomes necessary to cut the piles off below water surface and to build frame trestles on them the piles are in the proper position to receive the weights to be carried—that is, in the prolongation of the posts of the trestle, both vertical and inclined. This is not the case in bents with vertical piles, and when these are cut off the batter-posts have no direct support under them unless extra piles are driven for the purpose, ultimately adding 848 piles per mile, or 2 to the bent. It would be equally sensible to construct the framed trestle bents with four vertical posts. This will probably be realized when contractors are forced to burn up their old, rickety and broken-down drivers that have been in use for a quarter of a century. In a 3-pile bent with outside batter-piles the piles at the top are arranged in regard to the rails, as in the vertical pile bent. But whatever may be the height of the trestle bent its base is spread proportionally, and its stability is secured. A trestle with 3-pile bents of this kind has more bearing power, more stability, and costs less by $5478 per mile than a 4-pile bent as usually driven with all vertical piles. Whether 4- or 3-pile bents are used the outside batter piles should be used. These four types of trestle bents are shown in Fig. 36, (5), (6), (7), and (8), respectively; a mere glance at the drawings establishes the truth of the above comparisons. The X brace of 3 in. plank are used in all cases, and longitudinals should be used when the trestles are over 10 or 12 ft. in height. The decks, composed of stringers, ties, and guard-rails, are or may be the same in all cases. Fig. 36, (9) and (10), shows the elevation and plan for either type used.

105. Sometimes it is necessary or desirable to use pile foundations either of wood or iron, from an economical point of view, when the bed of the stream is of a rocky material, as on coral reefs or layers of marl or other soft stone, into which it is either very difficult or impossible to drive piles with or without shoes. In such cases holes can be drilled of the proper size, a fraction less in diameter than the piles and to the required depth, and piles can then be driven firmly into these. The depth need not be very great,—from 1 to 3 ft.,—as the only object is to hold the bottom of the pile in place, or small screw disks of iron can be fastened to the bottom of the pile, and by levers or other suitable machinery the piles can be screwed into the material to a sufficient depth. This would, however, be rather applicable to iron shafts or columns than to wood. Upon these suitable platforms are constructed. Such structures are frequently constructed for various purposes on the sea-coast. It must be remembered that timber cannot be used in such situations unless creosoted, as it would soon be destroyed by sea-worms.

106. Instead of drilling holes for piles, cribs heavily weighted with broken stone could be sunk on the bottom, and either timber or iron trestles could be built on these and well bolted to them. In this manner also trestles are sometimes built across rapid streams with rocky beds. A timber crib is framed, with pointed ends, 4 or 5 ft. wide, and somewhat longer than the bottom sill of a trestle bent, which for a trestle 20 ft. high would be about 24 or 25 ft.; this is sunk in place until one end is on the rock, and then, while suspended in a horizontal position, broken stone is forced under and around until a uniform and solid bearing is secured, as described in sinking cribs for piers in rocky beds. The crib is then filled with broken stone, the trestle framed on the crib, and secured to it by iron straps. Plank can also be spiked to the posts of the trestle, and the inclosed space filled with broken stone to add to its stability. Such trestles will stand a very great pressure in times of floods. The bents should be placed in a plane parallel to the direction of the current rather than perpendicular to the line of road.

ARTICLE XLV.

COST OF TIMBER TRESTLES.

107. TIMBER trestles should only be regarded as temporary expedients, required by considerations of economy or rapidity of construction, or sometimes from necessity; and it is generally expected to replace them by earthen embankments, iron viaducts or trestles, or by masonry abutments, piers, and bridge spans. Such considerations should not be allowed too much weight in designing and constructing such structures, as experience proves that they often remain for many years, repaired and renewed from time to time; often this is not done until some disastrous accident or wreck occurs, and even under favorable circumstances the substitution is slow and gradual and many years elapse before it is entirely made.

108. When framed trestles are to be built on the piles of former existing pile trestles, a record of the original pile-driving would be an important paper. Unfortunately, such records are not usually made, and if made not preserved. A pile trestle suitable to carry the comparatively light engines and loads of 20 years ago might not be strong enough to carry those of the present day, unless the piles were originally driven to the depths and resistances previously mentioned as safe under the present heavy loads. The writer has generally kept a pile recorder and inspector with every driver. The records showed the number of blows, the length of each pile, its larger and smaller diameter, and its penetration at each blow for the last 4 or 5 blows, and the depth driven in the soil, the number of feet cut off after driving, the extent of brooming or splitting if any existed—in other words, a complete history of each pile. All that is needed is an honest young man, who will obey instructions. The satisfaction in such a record is worth all it costs; it will make known the weak points in a structure, and will enable the chief engineer to have a full knowledge of the character of the work done, and to order additional piles to be driven in certain places, if the record should seem to require it. Daily records of the progress of the work should be made and forwarded at stated intervals, with nature, causes, and extent of delay. This is applicable to all kinds of work, and at all times, requires but little extra labor on the part of resident engineers and inspectors; makes these young men alert and observant, and keeps the chief engineer fully posted on matters of detail and importance, and in addition such records are of great value to the company for future reference and use. The want of such records often costs the company many thousands of dollars, while the cost of having them will amount to only a few hundred dollars. A striking case of this kind occurred in 1886–87, when the writer was chief engineer of the Mobile & Birmingham Railway. A railroad had been constructed for 60 miles out of Mobile to the Tombigbee River and was operated for several years, but owing to the inability of the company to renew the trestles the aggregate length of which was about 5 miles, the road had been abandoned for several years. This road was constructed in 1871–72. Upon assuming charge of the rebuilding it was naturally concluded to cut off the old piles below low-water line and to build framed trestles upon the old piles; there were no records made, or certainly none were filed among the papers of the company that were preserved. This want of information caused some hesitation, but, believing that the road had been originally constructed in a thorough manner, a large force of hands was employed at several different points to cut off the piles; lengths of trestles were measured, bills of material made out, and partial contracts made. In the mean time, all of the old piles on several trestles had been cut off; considerable time and money had been spent on these, when unexpectedly the writer was called to examine the trestles at one or two points, and to his surprise he found that in many cases where an excavation had been made to get below the line of constant moisture the piles had been entirely undermined at a depth not exceeding 3 or 4 feet. Examinations were made at other points to find the points of the piles, which was done without difficulty. It became necessary to change all plans and contracts, and to repile the entire distance. Information obtained from old residents pointed to the fact that originally no special inspection of the driving had been made, and the contractors had been left to do very much as they pleased; that if a pile was inconveniently long it would be cut in two pieces and hit a few blows, and that the trestles had been to a large extent built in this way. The business on this 60 miles was very small; only light trains drawn by light engines had been run over it, and these poor trestles were able to stand the small loads brought on them. After discovering this condition of things no confidence could be placed in the bearing of the piles in any of the trestles. To change orders at the mills, to make contracts for driving piles, and to change contractors, cost the company a good deal of money and loss of time, involved the company in suits; and in some instances these were decided against the company, notwithstanding the fact that it had been provided in the contract that the right to change plans or reduce quantities of material was specifically reserved. Had a regular pile inspector been employed such work would not have been allowed, and the chief engineer would have been posted in regard to the matter. Had this road been completed in the beginning and heavy engines and trains run over it, many of the trestles would inevitably have given way, and life and property would have been destroyed.

One trestle on this road, about 1300 ft. long, was built with black cypress piles; and although these piles were rather small, not averaging more than 9 or 10 ins. in diameter, had been exposed for more than 15 years, many of them were found in a fair state of preservation, and had been used continually for hauling logs to a saw-mill in the neighborhood—a small locomotive running over it constantly, drawing flat cars loaded with lumber. All of the pine timber originally used had entirely rotted and in the main fallen down, though in places some sound timber could be found.

Some engineers do not cut off the piles and build framed trestles, but repair and renew the trestles by driving new piles when necessary.

109. A comparative estimate will now be given of the quantities and cost of framed trestles and pile trestles of the same height. We will assume a trestle, the total height of which is 21 ft. 9 ins., the framed trestle resting on mud sills, bents 12½ ft. from centre to centre; we then have, for a length of 12½ ft.:

2 guard-rails	6 × 8 ins.	× 12½ ft.	100 ft.	B. M.
12 ties	6 × 8 "	× 9 "	432 "	"
4 stringers	6 × 14 "	× 12½ "	350 "	"
1 cap	12 × 12 "	× 10 "	120 "	"
2 posts	12 × 12 "	× 19 "	456 "	"
2 batter-posts	12 × 12 "	× 19⅝ "	471 "	"
1 sill	12 × 12 "	× 19½ "	234 "	"
6 mud-sills	12 × 12 "	× 5 "	360 "	"
2 diagonal braces	3 × 12 "	× 26 "	156 "	"
			2679	

Total timber in a length of 12½ ft. 2679 ft. B. M. at $26.00= $69.65. The timber in the pile trestle is the same as in the frame trestle diminished by the posts, bottom sill and mud-sills; or, in this case, 1158 ft. B. M. at $26.00 = $30.10, and increased by 4 piles 50 ft. long = 200 lineal ft. at 20 cts. per foot = $40.00, or the total for 12½ ft. = $70.10, a difference of 50 cts. nearly in favor of the framed trestle. The cost of the iron is generally included in the price for framing, and is practically the same in both cases. In the first case the cost per foot of length is $5.57, and in the second $5.61. The above prices are the actual contract prices paid for about 5 miles of trestle in 1886. For timber framed in trestles the price varies from $22.00 to $30.00 per 1000 ft. B. M., whereas the cost of driving and furnishing piles varies from 20 to 40 cts. per lineal foot; and although in the example above given the cost of the two trestles are practically the same, at different prices between the limits above given there might be a wide difference in the relative costs. In general, it can be stated that the cost of framing timber in trestles including the iron will be from $8.00 to $10.00 per 1000 ft. B. M.; and this added to the ascertained cost of the timber, delivered and piled at some convenient point near the site of the structure, will give a close approximation to the cost of the completed structure. And in the same way we may say that 10 cts. added to the cost of piles per foot of length when delivered at the site of the structure will be somewhere near the total cost per foot of piles when driven; but this is more liable to variation than the cost of framing, as the handling of piles costs more in some cases than in others: if, for instance, piles can be delivered on the banks of a stream and then floated to the driver, the handling will cost but little; if, on the contrary, they have to be dragged over swampy ground for any distance, or a track has to be laid and the piles hauled on trucks, the cost will be greatiy increased. The labor of cutting off piles to receive the caps is generally included in the cost of framing; and unless the same party does the driving and the framing, the party driving the piles will add something for the cutting off, and the party doing the framing will not make any deduction on that account. And in either case the amount of work to be done in any one place materially affects the cost of the work, as camps have to be established, shanties constructed, supplies of provisions secured. Every move of camp, transfer of drivers, machinery and tools add to the cost of the work. The cost of the work, however, will be found within the limits above given—that is, for framed work from $22.00 to $30.00 per 1000 ft. B. M., and from 20 to 40 cts. per lineal foot of piles driven; the superior limit also including the cost of cutting off piles under water. The weight of round iron for each foot of length is as follows:

Diameters	$\frac{1}{2}$ in.	$\frac{5}{8}$ in.	$\frac{3}{4}$ in.	1 in.
Weight per foot	0.66 lb.	1.00 lb.	1.5 lbs.	2.64 lbs.
" nuts and heads	0.20 "	0.36 "	0.7 "	1.75 "
" two-plate washers	0.20 "	0.20 "	0.2 "	0.31 "
Total weight	1.06 lb.	1.56 lbs.	2.4 lbs.	4.70 lbs.

From this table it is easy to calculate the amount of iron and cost per bent and one span of $12\frac{1}{2}$ ft. in length. Some engineers use a little heavier bolt than others, but the following will be ample for any trestle. Wrought spikes will weigh from $\frac{1}{2}$ to $\frac{3}{4}$ lbs. per spike 10 ins. long and $\frac{1}{2}$ in. square under head.

The bill of iron for a bent of trestle and one span of $12\frac{1}{2}$ ft. in length will be as follows:

6 bolts for fastening guard-rails to cross-tie, $\frac{1}{2}$ in. $\times$ 1 ft.	3.96 lbs.
Nuts, heads, and washers for the same	2.40 "
18 spikes fastening guard-rail to ties, $\frac{1}{2}$ in. $\times$ 10 in.	9.00 "
24 " " ties to stringer, $\frac{1}{2}$ in. $\times$ 10 in.	12.00 "
2 bolts for fastening stringer to cap, $\frac{3}{4}$ in. $\times$ 2′ 4″	3.60 "
Nuts, heads and washers	1.80 "
8 bolts for bolting stringers together, $\frac{5}{8}$ in. $\times$ $1\frac{1}{2}$ ft.	12.00 "
Nuts, heads, and washers	4.48 "
12 bolts for fastening X-bracing to posts, cap and sill, $\frac{1}{2}$ $\times$ $1\frac{1}{2}$ ft.	11.88 "
Nuts, heads, and washers	4.80 "
Total iron for each $12\frac{1}{2}$ feet of trestle	55.72 lbs.
In case of pile trestle add 4 drift bolts, 1 in. $\times$ 2 ft.	21.16 "
And if longitudinal bracing is used, 4 bolts $\frac{1}{2}$ in. $\times$ $1\frac{1}{2}$ ft.	3.96 "
Or total iron	80.84 lbs.

If straps $\frac{1}{2}$ in. $\times$ $1\frac{1}{2}$ in. $\times$ 2 ft. are used instead of drift bolts,

8 straps, 5 lbs. each	= 40.00 lbs.
16 bolts $\frac{1}{2}$ in. $\times$ 15 in.	= 13.12 "
Nuts and heads	= 3.20 "

Total..........56.32 lbs. less drift bolts 21.16 lbs.	= 35.16 lbs.
Total	116.00 lbs.

In the three suppositions above, the amount of iron per foot of length of trestle is respectively 4.45 lbs., 6.47 lbs., 9.28 lbs., and the cost at 5 cts. per pound is respectively $22\frac{1}{4}$ cts., $32\frac{1}{3}$ cts., $46\frac{2}{5}$ cts. These differences seem small, but they amount to a considerable sum in a mile of trestle. The second costs \$532.40 more than the first, and the third costs \$1325.12 more than the first, and \$792.72 more than the second for each mile. It is often necessary to use the straps, notwithstanding the increased cost, and in addition when a trestle is to be renewed the straps can be used over again, whereas drift bolts can rarely be used a second time; and for this reason straps, though more costly at first, will prove ultimately to be the most economical, and in addition lessen the labor and cost incident to repairs and renewals. Instead of using screw bolts for the longitudinal and diagonal braces, either wrought or cut spikes are frequently used; ultimate economy will result by using the bolts, besides other advantages. This subject has been discussed more in detail than it apparently deserves; but the importance of such things is apparent to both engineer and contractor, and it will often be found that a little knowledge on these seemingly small and unimportant subjects will be of inestimable value to the engineer. Bolts are simply used as a rule without any regard to the actual diameters required—1-in. bolts used where a $\frac{3}{4}$-in. bolt would be sufficient, and $\frac{3}{4}$-in. bolts used where a $\frac{5}{8}$ or $\frac{1}{2}$-in. bolt would answer every purpose. A glance at the foregoing table shows that $\frac{3}{4}$-in. bolt with head, nut, and washers weighs 2.4 lbs., whereas an inch bolt weighs with nut, head, and washer 4.7 lbs. for a bolt 1 ft. long, that is, about twice as much.

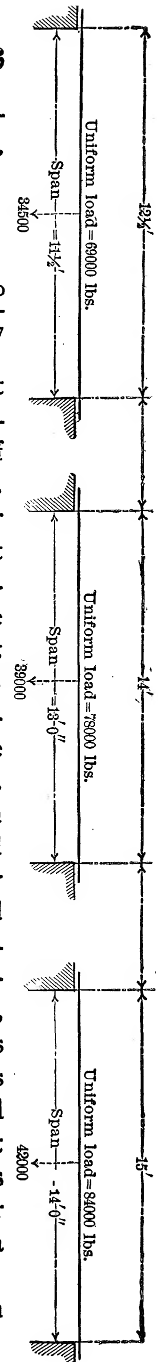
110. When the bents are placed farther apart, or when the spans are longer, the number of bents to the mile are less, thereby saving material; but the length of the stringers required causes an increase in their dimensions or in their number or in the timber required in the straining-beams and struts, and some increase in the number of bolts. With any given height of trestle it would be an easy matter to determine the economical length of span to use, as the dimensions of the timber in the bents themselves are practically the same in spans from $12\frac{1}{2}$ ft. to 25 ft. in length. In the shorter spans there is always a large excess of timber in the bents above that actually required to carry safely the loads, as the four posts together have a total cross-section of at least 528 sq. in. area, or a safe resistance to crushing at 500 lbs. per sq. in. of 264,000 lbs., whereas the total load, rolling and fixed, would not exceed 90,000 lbs., and for the 25-ft. span the total load will not exceed 140,000 lbs.; therefore up to this limit, economy would justify the long spans rather than the short ones. But as the spans increase the supports or foundations of the bents must be stronger, and in pile trestles more piles to the bents would be required, unless the material into which they are driven is firm and compact. Engineers have apparently settled on certain lengths of spans without considering the question of economy, and we therefore find the standard spans for a single-story trestle either $12\frac{1}{2}$ or 14 ft., occasionally 15 ft., and for trestles of two or more stories 20- and 25-ft. spans, the dimensions of stringers for the spans seem to be determined in the same arbitrary way; and we find for the spans $12\frac{1}{2}$, 14, 15 ft. the following dimensions respectively: 4 pieces each 6×14 ins., 7×15 ins., and 8×16 ins. Applying the formula $mWl = nfbh^2$ to these three cases, W being the equivalent centre load, or one half of the uniform load on the clear spans, $l = 11\frac{1}{2}$, 13, and 14 ft. respectively, and the load 6000 lbs. per ft. is equal to 8625, 9750, and 10,500 lbs. respectively; $f = \frac{1}{6}$, $b = 6$, 7 and 8 ins., and $l = 138$, 156 and 168 ins., respectively. We find $h = 17\frac{1}{4}$ ins., 18 ins. when $f = 1000$ lbs., and 14 ins., $14\frac{3}{4}$ ins., and 15 ins. respectively, when $f = 1500$ lbs. We may then conclude that theoretically the assumed dimensions of the stringer are sufficient. For the spans 20 and 25 ft. the stringers can be of the same dimensions as given above; the dimensions of the struts or rods used in bracing them being determined by the lengths of stringers supported directly by them, as explained in paragraphs 33 and 70. We will now determine by formula the depth of stringer required when six string-pieces are used instead of four, as above considered. In this the total load (one half of the total uniform load) divided by six will give the equivalent concentrated single load at the centre as illustrated in the following diagrams for the three different lengths of span. W then in the formula will $\frac{34,500}{6}$, $\frac{39,000}{6}$, $\frac{42,000}{6}$, equal respectively to 5750, 6500, and 7000 lbs., the value of all other quantities as above, from which the values of h or the depth of stringer in inches will be respectively (for $f = 1000$) 14 ins., $14\frac{3}{4}$ ins., 15 ins., and (for $f = 1500$ lbs.) $11\frac{1}{2}$ ins., 12 ins., 13 ins. Therefore we can use stringers under each rail composed of two pieces each 6 ins. $\times$ 14 ins., 7 ins. $\times$ $14\frac{3}{4}$ ins., 8 ins. $\times$ 15 ins., or three pieces 6 ins. $\times$ $11\frac{1}{2}$ ins., 7 ins. $\times$ 12 ins., 8 ins. $\times$ 13 ins. for the spans respectively of $12\frac{1}{2}$, 14, and 15 ft., centre to centre of bents with equivalent strength in each case, and either of these for the 20 and 25 ft. spans, when properly trussed as already explained.

111. Shall we use then the standard $12\frac{1}{2}$-ft. spans for a single-story trestle or 15-ft. spans? We have seen in paragraph 109 that the cost of a 12-ft. span of framed trestle is \$5.57 per foot of length, and that the amount of timber was 2679 ft. B. M. in a span of $12\frac{1}{2}$ ft. Deduct from this the guard-rails, cross-ties, and stringers, amounting to 882 ft. B. M. there remains 1797—to which add

4 stringers 8 in. × 15 in. × 15 ft. = .	600	
15 cross-ties 6 in. × 8 in. × 9 ft. = .	540	
2 guard-rails 6 in. × 8 in. × 15 ft. = .	120	1260
We have total timber in a span of 15 ft. in ft. B.M. . .		3057

which at \$26 per 1000 = \$79.48, or cost per foot of length \$5.30, a saving of 27 cts. per foot, equivalent to a saving of \$1425.60 per mile of trestle. If six string pieces 8 ins. × 13 ins. × 15 ft. = 780 ft. B. M., or 180 ft. more timber, equal to \$4.68 per span, or 31 cts. per foot, making the cost in that case \$5.61 per foot, which shows that the substitution of 8 × 13-in. stringers is not an economical use of stringers, as might have been expected; but it may often be difficult to secure 8-in. × 15-in. stringers of clear heart, whereas the 8-in. × 13-in. stringers could be secured. But this is a little more expensive than the spans of $12\frac{1}{2}$ ft. long with 6-in. × 14-in. stringers; and if this size of stringer can be obtained it would be preferred, as there would be less pressure on the supporting material, whether mud-sills or piles. The above considerations and principles have a very much more important application when deciding upon the economical relations of piers and length of spans in long bridges, as in such cases the economical length of span is a matter of very great importance. But in this place it will be sufficient to say that as a general rule in low structures the spans should be short with many supports or piers, and in high structures the spans should be long and with few piers or supports. This will be considered more fully in a subsequent chapter.

112. It may, however, be stated here that in



very high trestles it will be better to use spans from 40 to 50 ft. or more, and to construct piers or double bents similar to the timber piers described in paragraph 28. This can only be decided by a careful estimate of cost, including the extra precautions required to secure safe foundations. The principles involved have been fully discussed in the preceding paragraphs.

113. As a general rule contracts provide for payments on the basis of so much per 1000 ft. B. M. framed trestles, and so much per lineal foot of piles driven, and sometimes in addition so much per pound for iron used. Sometimes, however, the contract is so much per lineal foot of completed trestle. This last method possesses many advantages, as in this case there can be no dispute as to the final settlement. The work shows for itself; either party can measure the length. In other cases questions may and do arise every month; the contractor is not satisfied with his estimate, complaints are made, and extra bills presented. It is difficult to provide for every contingency in contracts—whether the lengths of posts mean from end to end of tenons, or whether the tenons are to be excluded; how the cut-off ends of piles are to be paid for, and packing blocks between stringers; excavations required for framed trestles resting on mud-sills, excavations for box-culverts, baling and pumping out water after rains, and many things that may arise during the construction of extensive works. It is true that these things can be provided for in the contract, but however fully and carefully the contract may be drawn such questions will arise, extra bills of innumerable kinds will be presented, and in the end suits will be brought which will often be decided in favor of the contractor, even when they have no shadow of a just claim. The contract based on the foot of length is open also to some objections, and particularly if the engineer does not know by careful estimates the relations between the costs of the actual quantities of material and the price per foot, as the contractor will certainly on his part put the cost per foot at the highest possible figure, making his estimates on very liberal allowances for quantities and contingencies.

114. In order to avoid waste of material local customs should be examined into. In large saw-mills doing a regular business, certain definite lengths of lumber as well as sizes are in current demand, either for local use or for shipment to different and distant places, logs are cut so as to yield these lengths and sizes; and all bills of lumber that cannot be fully adjusted to these will entail either extra cost at the mills or waste in the works, for which the company will have to pay. If the common run of the square timber and plank, scantling, etc., is in lengths of even numbers, such as 12, 14, 16, 18, and 20 ft., it will be found economical to make the bill of lumber for any particular structure to correspond as far as possible. To specify that the posts of a trestle bent should be exactly 18 ft. $3\frac{1}{2}$ in. when an 18-ft. post would do as well is simply to add to the cost. Where definite lengths must be obtained it cannot be helped. Square timber such as 12 ins. $\times$ 12 ins. is used for stringers by some engineers, owing to the difficulty and cost of obtaining such sizes as 6×14 ins. or 7×15 ins. Either using shorter spans or using built beams for the longer ones, or as before mentioned the number of pieces can be increased, thereby decreasing the depths to 12 inches. These matters are merely suggested as useful hints, and to suggest the advantages to be derived from allowing slight variations in designs, rather than to follow some stereotyped and iron-clad conditions simply because somebody else has followed them before—always bearing in mind that strength, suitableness, and durability are the first requirements; but obtain these conditions at the least cost and in the least time.

115. It is often necessary to cut piles off below water surface; this may be required at any depth below the surface from 3 to 20 ft. or more, as when cribs or open caissons are to be sunk until they rest on the piles. There are three methods of doing this, 1st. By the use of professional divers. This is an expensive and slow process, as at best they can work only a few hours a day, and they charge high for their services. The diver's suit consists of a water-tight canvas suit of clothes, which covers and fits the body from the neck to the ankles. Around the wrists and ankles this is bound tight to the skin by strong rubber bands. Over his head a copper helmet is placed, in which are thick glass plates called bull's-eyes; this fits over his shoulders and is fastened to the water-proof suit by proper clamps, rubber bands being placed between. Connected with this helmet is a long, flexible tube or hose, which connects with the helmet at one end with a valve opening inward, and at the other with an air-pump. The helmet also has an escape valve for foul air opening outward. The bull's-eyes are protected by a metal netting to prevent danger of breaking; these should have water-tight valves, which the driver can close if required. To enable him to sink in the water the soles of the shoes are made of lead, and in addition lead weights are fastened to his breast and back. There is an opening in the helmet, which is closed by screwing on a cap just as the driver is ready to descend. The helmet is made of copper. As soon as the cap is adjusted the air-pump must be started, very slowly at first, but more rapidly as the diver descends. A tender, as he is called, holds the hose in one hand and a rope securely tied to the body of the diver in the other, and he pays out these as the diver descends or moves about on the bed of the river. It requires two men at the pump—one at work and the other resting; these relieve each other at short intervals, and they should turn the crank at a uniform rate, so as to keep a constant pressure of air in the helmet. The diver signals by jerking the rope once, twice, or three times; these have some understood meaning, such as more air, less air, or to lift him up, and so on. As he rises toward the surface the pump is worked slower and slower, and when the cap is removed it stops. Divers can work in depths of water to 75 or 80 ft., but only for a very short time at the greater depths. Owing to the cost of the diver's services piles are cut off by saws worked by machinery from above.

116. A simple arrangement for this purpose is to fasten a cross-cut saw to the bottom of a frame which is connected to a rod suspended from a bolt attached to a frame constructed on a barge; the saw being adjusted to the proper depth, a swinging motion is imparted to it by men on the barge or platforms from

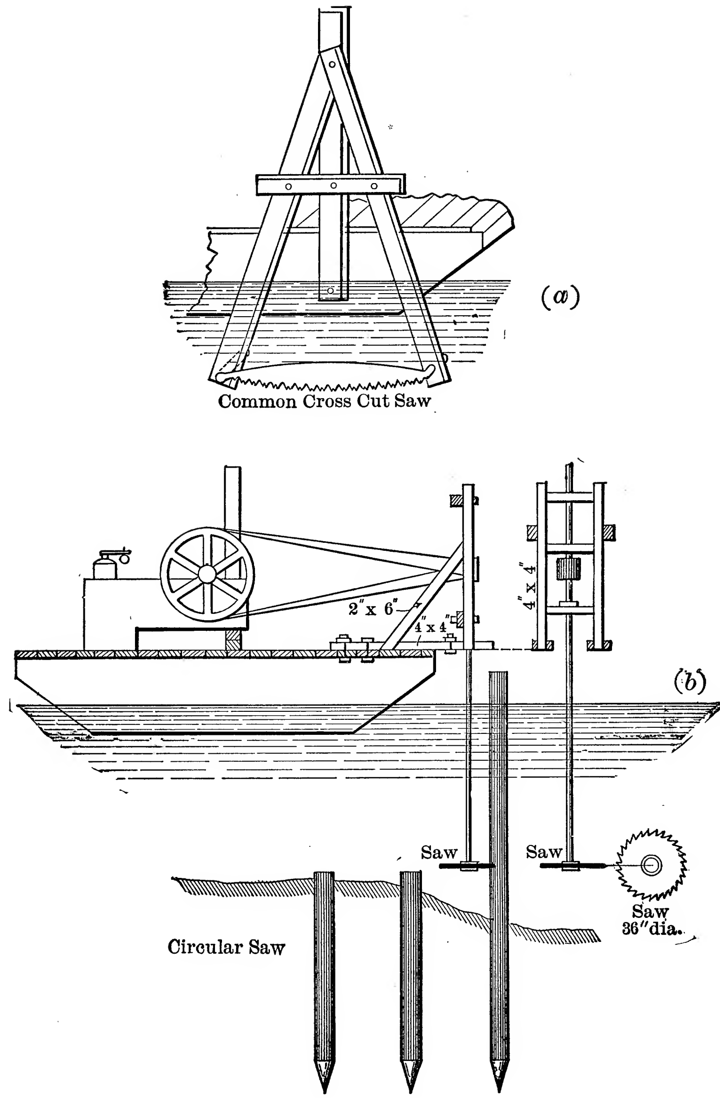


FIG. 37.—ORDINARY MACHINES FOR SAWING OFF PILES UNDER WATER.

[*To face page* 247.]

above, and as it enters the pile it is pressed forward by a lever attached to the bottom of the lower frame. When one pile is cut off, it is moved to the next, and so on. Where there is no great current, or no ebb and flow of the tide exists, good progress can be made and good work done by this method. While sawing the boat must be kept level, unless the frame above admits of the suspending-rod sliding up and down. Where a large number of piles are to be sawed off, the following arrangement is used (see Figs. 37 and 37 (*a*)).

117. A frame is constructed on a barge, or, better, a floating pile driver. A long iron shaft carrying a horizontal circular saw is so suspended and connected in the leads that it can be turned and at the same time raised or lowered in the leads; a band wheel or drum is connected with the shaft; the power band connects this with the drum of an engine. When the power is applied the shaft and saw are made to revolve rapidly. The saw is adjusted to the proper depth, and started; the pile is cut off in a few seconds. If there is no strong current, any number of piles can be cut off in a very short time. The only difficulty in a current arises from the difficulty of holding the barge steady. This can be easily controlled. In a tidal stream the depth of the saw has to be changed more or less rapidly as the tide ebbs and flows. To regulate this an accurate tide-gauge must be placed in some protected place, where it can be easily observed either by the foreman on the barge or by an assistant; a corresponding scale is also placed on the leads, adjusted to the plane of the top of the shaft, or some well-defined mark on the shaft. The reading on the scale and gauge are taken simultaneously at the commencement of the sawing, and afterward the saw is raised or lowered $\frac{1}{4}$ in., $\frac{1}{2}$ in., 1 in. from time to time as the tide falls or rises. With proper care and precaution a large number of piles can be cut off at practically the same elevation. Upon these the open caisson, or crib, or other structure, can be lowered. See Figs. 37 and 37 (*b*).

118. In driving piles over a space to be occupied by the structure, the outside piles should be driven so as to enclose a space several feet larger than the actual base of the structure, or equal to that covered by the platform or bottom of the caisson, as explained in paragraphs 20 and 21, and shown in drawing Figs. 26 and 26 (*a*).

119. As a rule, a structure thus sunk on the piles simply rests on them, the weight holding it in place; and although it is desirable to sink such a structure exactly in its true position, a few inches one way or another out of line or distance is not a matter of much moment in most cases, as the masonry required to sink it should be a few inches larger than actually required; and when the structure is finally resting firm and true, an offset can be made on the top of the masonry so as to place the structure in its true line and distance. The little excess of masonry thus used is far less expensive than that of repeated raising and lowering the caisson. And unless strong staging is constructed around the caisson, and it is suspended and lowered by long rods with threads and nuts, it is almost impracticable to lower a caisson absolutely in a desired position. Such staging and apparatus are expensive, the lowering is slow and tedious. If necessary, do these things; but when not necessary, such useless refinements will do to talk about, but must be paid for by somebody. Contractors will always make the company pay dearly for it. The driving of a few extra piles is far better and less costly, a small margin in the size of the platform being allowed. This is, in fact, necessitated by the requirements of its construction.

120. Cases may arise where it becomes necessary to prevent any tendency to slide off of the piles. In such cases timber strips can be bolted to the bottom of the platform, projecting downward between and below the heads of the piles, which will hold the structure in place. And in some cases iron pipes $\frac{3}{4}$ or 1 in. in diameter are built in the caisson, extending through the bottom; and when the caisson finally rests on the piles, long spikes or pointed drift-bolts can be dropped on the heads of the piles and driven into them by blows from above. But unless a grillage is constructed with small square openings in it, so as to guide and hold the piles in certain positions, the pipes would be as likely to miss the piles as to rest on them. Where such precautions are not necessary the exact positions of the piles are not of much moment; but a reasonable effort should be made to drive them in rows at specified intervals, such as $2\frac{1}{2}$ ft. from centre to centre, and no great error in position should be allowed. This cannot always be discovered until the piles are cut off, as when freed from wedges, bars, etc., they are apt to spring more or less.

121. In driving piles for trestle-work, it is important that the piles in each bent should be in line with respect to each other, and also that the piles in the different bents should properly line up with each other, for appearance sake, if nothing else. The difficulty of driving piles in exact line is doubtless very great and often impracticable, but it can be done much better than is often the case; and the piles have to be sprung into position by the application of a great force. This necessarily bends the pile, or that portion of it above ground, thereby putting it in an unfavorable position to carry heavy vertical loads. Often they are so far out of position that they cannot be sprung or pried into position, and consequently the cap rests on only about one half of the pile. These conditions often result from inexcusable carelessness. Proper care is not taken to set the pile or to hold it in the beginning when it can be controlled, but after it has penetrated to a considerable distance in the soil, and out of plumb or position, desperate efforts are made to force it back, which will then be only partially successful, if at all, and doubtless piles are seriously crippled or even broken below the ground in many cases. If the pile is properly pointed, head cut square, and set straight, and properly controlled by wedges or levers until it penetrates well into the surface, it will be easy to keep it straight to the finish. If a pointed pile strikes roots or even small bowlders or other narrow obstructions, it will inevitably veer out of position, and no power can prevent it. It must be then put back the best possible, and in some instances new piles have to be driven. As previously mentioned, it is easier to keep a blunt pile straight than a pointed one. In alluvial and the softer soils the pile should not be sharpened. It may sometimes be necessary, however, in the firmer, stiffer soils.

122. It would be better to mark with a peg or stake the position for the point of every pile; this takes time and labor and the dragging and lifting of piles and heavy timbers will destroy many of them, but enough will remain to prevent any serious error in alignment or position. At any rate a peg should be driven to mark the centre of every bent, and for a few bents at the beginning and at intervals of every 200 or 300 feet pegs should be placed for every pile. By this means, the piles can be lined by sighting, and the small and gradual errors can be rectified at short intervals, and by fastening battens at intervals on the piles already driven the leads of the driver can be properly lined for long distances. On shore, with set lines of high stakes with strips of paper fastened to them, or better small flags, and occasionally placing the leads in exact position with the transit, a true line can be kept for long distances. This should always be done when driving across water.

123. It is better not to drop a pile after being lifted between the leads, unless it is so long that this is necessary to get the head under the hammer, as it is difficult to drop it in the exact position or in a vertical line. Sometimes it is necessary owing to its length to drop it in front of the leads, so as to let it penetrate as far as possible into the soil, and then move the driver forward; this requires great care.

124. During the driving the direction of the pile is controlled by short blocks of wood, wedges, and levers. The leads of the drivers have iron brackets bolted to them, which hold the blocks of wood. The pile is lifted in the leads, lowered and set in position at its point, forced into a vertical position, and the blocks are then placed in front and rear and wedged into position. This is done at one or two points in its height; then the driving commences: the wedges are loosened or tightened so as to keep the pile vertical, or these are omitted, and the piles held and controlled by levers handled by the men.

This imposes very hard work on the men during the entire time of driving. Either plan can be used, but the first seems to be preferred. In stiff, compact silt, or ordinary clay it will be found convenient to drive a short pile, which is then pulled out, and the longer pile let down into the hole to a depth sufficient to bring the head of the pile under the hammer. Long, heavy piles can be set more accurately in this way than by dropping them in front of the leads.

ARTICLE XLVI.

EMBANKMENTS OF EARTH ON SWAMPS.

125. As has been mentioned, timber trestles are to a large extent temporary structures, and it is expected to substitute iron trestles or embankments of earth sooner or later. This is also applicable to a considerable extent in building roads across extensive swamps; but here it must not be lost sight of that the rises in the rivers and streams intersecting them, and the flow of the tides, especially in cases of storms, cover these swamps to the depth of 3 to 6 ft., and that ample water-way must be provided. Therefore long stretches of trestle are necessary, which will constitute permanent important parts of the work. With this precaution it is intended to ultimately form earth embankments when material for the same can be secured in the necessary quantities and at a reasonable cost, and after the road has been constructed the construction trains can gradually dump dirt under and around the trestles until the embankments are completely formed. This will require a large amount of material, as the weight of the earth breaks through the matting or crust of roots and sinks to an unknown depth, but ultimately it will cease to settle, and a permanent embankment takes the place of the trestles.

126. The matting of roots, of the cane and undergrowth that grow so largely in these swamps, has sufficient strength to carry the weight of two or three feet of earth and a light construction engine and dump-cars; but this is its ultimate strength. Any increase of weight will break through. When, therefore, earth can be obtained from the neighboring elevations, these are staked out for borrow-pits, and tracks are laid from them to connect with the track of the main line of the road. The cross-ties are laid directly on the swamp. Trains loaded with earth are then run on to the main track, the dirt dumped on the swamp, and gangs of men raise the track with levers; the earth is thrown and rammed under the ties. When this embankment reaches a height of two to three feet, the crust breaks short off along the foot of the embankment; the embankment and track settle into the liquid mud underlying the crust. As fast as the material can be added it settles down; embankments several feet high in the evening will entirely disappear by the following morning, only to be filled again. This may continue for weeks, gradually settling more and more slowly, until finally it will practically cease, but in a greater or less degree will continue for months or years.

127. The depth to which this will reach below the swamp is probably not known, but must be very great—not less than 10 to 15 ft. This conclusion was reached by the writer in observing the effect upon the swamp on either side of the embankment. The swamp bulges up fully 6 ft. on either side, somewhat abruptly facing the bank, and sloping rather gently from this summit outward to the level of the swamp, the crust forming over the mound a smooth, uniform covering, this mass of material representing the displacement made by the earth of the embankment. The earth doubtless assumes a slope considerably steeper than its natural slope, probably not more than $\frac{1}{2}$ to $\frac{3}{4}$ to 1, which would fully justify the depth above stated. After the lapse of time the mound settles down to the general level of the swamp; this has been observed for miles. As a further proof of the great fluidity of the material and the depth to which the earth sinks, the effect upon the trestle approaches of bridges over the bayous and streams intersecting these swamps will be mentioned. The approaches to these bridges were built of pile trestles in lengths from 25 to 100 ft., and as the earth embankment was built up to the ends of these and pressed against the end piles, the entire trestle would be pushed forward, and this also pushing the abutments against the ends of the draw bridge so firmly as to prevent the draw from opening until the latch beams were moved backward; and this was repeated many times. The material was so soft that in walking on the swamp and failing to plant the foot on the roots of the cane a man would sink to his waist before getting support. Such was the material upon which 14 miles of road was constructed, and into which piles driven from 30 to 40 feet would support load. The material for such banks should be sand and gravel or sand alone; clayey soils would be apt to form mud, and be but little firmer than the swampy material itself.

128. Sometimes a layer of long logs or plank is first laid on the swamp so as to give a broad base for the embankment, and if broad enough it would keep the crust on the surface from breaking through; this answers well for support, but is probably wanting in steadiness, and a rapidly moving train tends to produce a wave-like motion. It is more economical than the first method, but cannot be considered as good or as safe.

129. These methods of embanking are very objectionable, as the track has to be raised as the earth is packed under the ties. The result is that the rails are badly sprung or bent, both in a vertical and a horizontal plane; the former being more objectionable than the latter, as the horizontal bends are the more easily seen and removed in part, if not entirely. A temporary trestle consisting of two short piles could be constructed of the proper height; this would carry a light train, and the earth embankment could be formed under and around it. It would cost something, but at any rate would save the rails. It is the plan adopted even on firm ground, where the material is hauled out by engine and cars. The trestles in this case are framed and a light rail is used. The cost, however, would be the same in the two cases, which would be practically offset by the extra labor required in rehandling the earth and raising the track.

130. A few general remarks on earth-work will be made in this connection. The earth-work on a line of road consists of embankments and excavations. After locating the line of the road and establishing the grade line the road is divided into sections, somewhat in an arbitrary manner—the length of the sections being so regulated that the material excavated may be sufficient to make the embankments within the limits of the section, or for some other reason, the average length of the sections being from one to two miles. The earth from the excavation is hauled by barrows, carts, horse-cars, or by the use of a locomotive engine, depending on the amount of work, length of haul, etc. Sometimes it is more economical to waste the material from the excavation on the sides or at the ends of the cut, and to make the embankments from trenches, ditches, or borrow-pits along the embankment; these matters are regulated by considerations that will not be discussed in this volume. Ditches along the embankments are necessary for purposes of drainage, and should be cut as straight and as regular as possible. A space called the berm should be left between the foot of the embankment and the ditch; the width of this space is regulated so that the prolongation of the plane of the slope of the embankment shall pass well under the bottom of the ditch on the berm side, which will require a berm of from three to six feet, according to the depth of the ditch. The side of the ditch should have a slope whose base is equal at least to its depth, so as to prevent caving in. The cuts are excavated so that the slopes will be one vertical and one horizontal, or one to one as it is called, and the width at bottom varies for a single track from 16 to 18 ft., so as to allow for side drains. The width of the embankments on top vary from 12 to 14 ft., and the side slopes one and one half horizontal to one vertical. Whether the embankments are made from cuts or trenches, they should always be started with the full width required at the bottom and maintained the full width to the top. A too common practice is to make a narrow core at first, and then to widen it out by dumping loose earth on the sides; the core being hauled over settles and compacts; the loose earth thrown on the sides sloughs off, and will not bond with it. And again, in making the filling a too common habit is to keep the embankment higher at the centre than at the sides. This is just the reverse of what it should be. Each layer should be a fraction lower in the centre. This rule should always be observed when the embankment is made in layers. Embankments made from cuts are generally built in one thick layer, of the required height of the fill; this is done by dumping the earth at the end of the embankment. The practice is still to keep the bank too narrow; it should be built of the full width from bottom to top. Broken stone, gravel, sand, or mixed earths make the best embankments. Clay makes a good embankment when put up dry and properly drained. All earth embankments will settle more or less, depending upon the character of the material used, and the manner in which the embankment is constructed. Clay and ordinary earth settle slowly, and to a considerable extent, and more than sand or gravel. It is not unusual to allow as much as ten per cent for settlement; that is, a bank ten feet high must be made eleven feet high on first construction. Low embankments that are made either by throwing the material from trenches, or by the use of barrows will require the full allowance for settlement. High embankments constructed by the use of scoops or drags drawn by horses, by horse-carts, and by engines and dump-cars, owing both to the time required in the construction, and also to the constant tramping and hauling over them, will settle to a large extent during construction, and will require but a small per cent. of additional height. The slopes of the cuts are liable to be washed into gulleys, and undermined by the flow of surface water running down the slope, or sinking into the soil and escaping along seams or through porous layers of sand or gravel. This can be greatly reduced by cutting surface drains on the up-hill side of the cut, or by surface drains on the slope made of timber, or by terra-cotta pipes imbedded in the slope, and emptying in the side drains at the bottom. Sodding the slopes or sowing grass-seed on them is also a remedy for this trouble, besides adding greatly to the appearance. In some cases these methods fail, and the slopes will cave in. In this case benches can be cut at different elevations, so as to break the slope; drains made on the benches will carry off the water. Foot walls can be constructed of masonry at the bottom of the slope; even when very thick they often prove of little value. All of these means failing, the material can only be removed as it falls. If the weight and amount of traffic were the same in both directions a straight and level line would be desirable; but in the direction of the heaviest traffic gentle inclines or grades are advantageous, as they aid in hauling long and heavy trains, and are of no serious obstruction on the return with the lightly loaded or empty cars. Grades also facilitate the drainage of the road-bed. Grades vary from 0 to 2 ft. per 100 ft.; the usual grades, however, vary from 20 to 52.8 ft. per mile; they should not be used on curves, trestles or bridges when not absolutely necessary.

131. The roadbed completed, when it can be economically done, on it should be placed the ballast, which consists of a layer of gravel or broken stone from 6 to 9 inches thick, upon which the cross-ties are laid; then between the cross-ties gravel or broken stone should be packed. The ballast gives firmness to the bed and serves also to drain off the water, thereby keeping the track dry, and consequently preserving the ties. As a rule, however, the ties are first laid on the earthen embankment, the rails laid, and the ballasting done afterward, when it can be hauled in construction trains and distributed more economically. The track is raised, and the ballast then rammed under the ties and between them, as before. When broken stone is used the ballast is built up to the top of the tie for its full length, with the proper slope on the sides. Sand and gravel also make good ballast; but sand, especially if very fine, makes a dusty road, and the grit deposits in the machinery, which causes friction and wear. Sandstone is apt to be pulverized, and has the objection just mentioned. In many sections of the country broken stone of any kind is hard to obtain, and the dirt ballast, so called, is used This simply means packing the dirt under and between the ties, so as to give firmness to the track. In this case drainage is provided by simply sloping the top, so that the surface at the centre of the track is level with the top of the tie and slopes gently on each side, so as to fall to the level of the bottom of the ties at their ends. The water is thereby drained off. But this kind of ballast is apt to work into the condition of mud during very wet weather by the churning motion imparted to the ties by a rapidly moving train. But it is the only kind of ballast used on thousands of miles of road in the Southern and Western States. It is not favorable for very heavy loads or very high speeds.

132. Cross-ties are placed generally at intervals of 2 to 2½ ft. centres; requiring from 2640 to 2112 ties to the mile. The depth of the tie is from 6 to 7 ins., the width from 8 to 10 ins., and length from 8 to 9 ft., 8½ ft. being about the average. These ties are hewn on two sides and the bark stripped off the other two. It is usual to place the ties somewhat closer together at the joints, these being between the ties form the suspended joint. In the other case the joint rests on the tie, a broad cross-tie being selected for this.

Cross-ties are generally made of white oak, post and chestnut oak, white or yellow pine, and sometimes of other woods.

133. The cost of the earth work varies somewhat, but, as a rule, the established price is: Earth, 16 to 20 cts.; hardpan, 30 to 35 cts.; soft or loose rock, 40 cts., and hard rock in large masses, 80 cts. per cubic yard, and when the material has to be hauled more than a certain specified distance an extra allowance is made, such as ¾ or 1 cent per cubic yard for each hundred feet of haul over 300 or 500 ft. Disputes often arise on this point on the final settlement from the indefinite manner in which this is expressed in the contract.

134. Cross-ties vary in cost from 20 to 50 cts. apiece, depending upon whether pine or oak is used, and upon the more or less abundance of the timber suitable for ties along the line of the road. These may be taken as extremes, the average prices, delivered and piled at intervals along the road, being 30 and 40 cts. respectively. Piles of ties should be formed by first laying two or three ties with intervals on the ground, then a solid layer at right angles to these, then a layer of two or three, and another solid layer, and so on to the height of a man's head. This mode of piling enables the ties to be easily inspected, favors the gradual seasoning of the tie, and adds greatly to the life of the tie. Solid or irregular piles of ties cannot be properly inspected. The general rule is to require the ties to be hewn to smooth surfaces on top and bottom. Careless work leaves gashes in the tie that admit and hold water, thereby hastening rot. One end of the tie is required to be cut square, though the other may not be. The ties are laid with the square ends on a line parallel to the centre line of the road.

135. On a road in Central America the writer used lignum vitæ and mahogany ties. This is mentioned more as an illustration of the use of that particular kind of material, which grows, regardless of its intrinsic value, in any particular locality, and which is often carried to the opposite extreme by using very inferior materials on account of the convenience and cheapness of obtaining them. Lignum vitæ would doubtless be an economical tie in the end, no matter what its first cost, but it cannot be obtained in large quantities. The life of a tie depends greatly on the use of good ballast and also upon its resistance to being cut into by the rail, as this induces rot and loosens the hold of the spike. This peculiar property is marked in the ties of the lignum vitæ kind.

136. As has been mentioned, the two proper and usual methods of building across swamps are first to dump the earth directly on the swamp, and continue doing so until settling practically ceases, or to prevent settling by floating platforms of plank, fascine-mattresses, or logs; and secondly, to drive piles and fill in to a great or less extent subsequently with earth. Either of these plans is good, but there have been fatal and serious blunders in building across swamps, by cutting canals with dredges along the road-bed and emptying the material on the road-bed. This is bad practice, for several reasons, 1. It cuts or breaks the crust formed by the matting of roots, which is the main reliance to prevent excessive settling and sinking of the bank. This has been done, and to remedy the blunder double rows of piles have been driven along the foot of the slope to hold the bank, but this failed—as this swampy material, especially when the crust is broken, has but little stability so far as lateral resistance is concerned, and the piles would spread outward at and near the top. The writer saw the above conditions on the road between New Orleans and Mobile. The subsequent labor and cost of securing a firm road-bed must have been enormous.

2. The material of the swamp, even if it can be held in place, is in the writer's opinion, unfit for use in an embankment. There is now being constructed a road-bed across these same swamps, in which an attempt is being made to do away with the first difficulty by cutting the canal at a distance of 50 to 150 feet from the road-bed. Whether this will be effective and fully remove the difficulty, probably is yet to be determined. The material thus dredged is hauled and used in the embankment; the second difficulty then still exists, and it is doubtful whether such an embankment will ever prove satisfactory. It is probable that a temporary trestle, built with two pile bents, and subsequently filled in with some more stable material, such as sand or gravel, would prove ultimately more economical and satisfactory. The method is certainly an improvement as compared with the preceding one just described.

An interesting instance of this subject is the observed settling of the peaty soil in Holland. An embankment of sand 8 ft. high, giving a load at base of 800 lbs. per square foot, compressed the peat underneath to two thirds its bulk; final condition of stability was only attained after two years. The embankment afterward carried safely the railroad trains. The soil was covered with a fascine-mattresses, to distribute the pressure and prevent the sand sinking into the soft silt. In constructing a station yard, a trench 15 ft. wide and 15 ft. deep was excavated around the space and filled with sand, and a 3-ft. layer of sand spread over the entire space; and for the station building itself a pit, 15 ft. deep and of horizontal dimensions each way 20 ft. greater than that of the structure, was excavated and filled with sand; the sand, as has been mentioned, distributing the pressure over the entire surface of the excavation.

The theoretical discussion of the stability of earth under pressure as found in Rankine is very pretty, even if not true, and easily reduces to the well-settled theory of fluid pressure. In Fig. 38, *ABCD* is the cross-section of the excavation in the

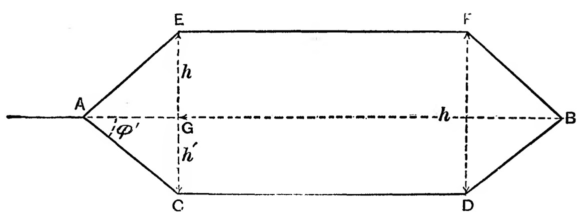


FIG. 38.

soft material, and *AEFBDCA* is a cross-section of the proposed embankment or other structure to be built. Making $GE = h$, $GC = h'$, $\phi' =$ angle of repose of soft material, $w' =$ weight of one cubic foot of the soft material, $w =$ weight of a cubic foot of the sand, gravel, or stone used in the structure, $k' = \frac{1 - \sin \phi'}{1 + \sin \phi'}$. Then *GC*, the depth to which the pit is to be excavated, $= h' = \frac{hwk'^2}{w' - wk'^2}$, for a fluid $\phi' = 0$, and the formula becomes, since $k' = 1$,

$$h' = \frac{hw}{w' - w} \quad \text{or} \quad w(h + h') = w'h',$$

which simply means that the structure will sink until the weight of the displaced soil is equal to the weight of the structure itself, which is the law of fluid pressure; for semifluids or firmer earths this is modified by the value of ϕ', the angle of repose of the material. Such formulæ should be of course used with precaution and a large factor of safety, and can only be regarded, as was mentioned in discussing the subject of retaining walls, as a very ingenious and masterly extension of the theory of fluid pressure to that of earth pressure.

Those cases in which a firm stratum is underlaid by a soft material, such as mud or quicksand, involving as they do many difficulties and requiring special methods of construction, will be classed under the head of difficult foundations, and will be discussed in the next part of this volume. And similarly, where the soft material overlies a firmer stratum, where piles are not used, although there are but few such cases in which piles would not answer every purpose, and in general be economical; but often they would be unsuitable, expensive, and undesirable.

PART THIRD.

ARTICLE XLVII.

FOUNDATIONS—(CONTINUED).

DEEP FOUNDATIONS.

1. HAVING considered what may be called ordinary foundations, including timber trestles and pile trestles, and in part first masonry and masonry piers from the foundation-beds to the bridge seats, we will now explain and discuss those foundations requiring more costly and difficult methods of construction. For convenience, foundations were divided into two parts, that portion from the foundation-bed reaching to or nearly to the surface of the ground, and that portion above and extending to the bottom of the superstructure; these together are commonly known as the substructure. To complete this portion of the subject, it only remains to describe certain unusual methods of reaching the foundation-bed, where great depths below the water or earth surfaces have to be reached. These methods, disregarding the materials used, which may be either wood or iron or both combined, may be divided into two classes. 1st. Where the desired depth is reached by simply dredging the material from the interior of a large timber or iron box or cylinder, suitably and strongly constructed for the purpose, and forcing the structure to sink against the exterior friction on its sides, by sufficient weights or loads superimposed. Structures of this class are called either open caissons, or more commonly cribs. 2d. Those methods in which timber or iron boxes or cylinders are constructed with one or more air-tight compartments, except that they are open at the bottom.

This part of the structure is called a pneumatic caisson, and upon this can either be constructed cribs of a greater or less height, on which the masonry rests, or this latter may rest directly upon the roof or deck of the caisson.

2. The first or crib method will now be considered. When constructed of timber, the crib is composed of four double walls of timber, enclosing a space of the proper horizontal and vertical dimensions; the two walls of each side may be built solid and connected together by horizontal struts and ties, or they may be built somewhat open and similarly connected. These walls near the bottom, and for a varying height, are constructed with V-shaped sections, coming together at the bottom edge, thereby forming a cutting edge, and opening out gradually to a width of 8 or 10 ft., at a height of about 8 or 10 ft. The outer wall has a batter or slope outward and downward, varying from a few inches to several feet, the inner wall is constructed to a slope of 45° or less. This lower section of the crib may be built solid with large timbers, or it may only be strongly braced with cross-timbers. Upon the top of this bottom section the two walls are built up vertically and parallel, or the outer wall may have a slight batter of about one half inch to the vertical foot, and the two properly tied together. The object of the space between the two walls is to give strength and stiffness to the sides of the crib and at the same time to supply sufficient space for the weight required to sink the crib, which weight is generally either gravel, broken stone, or concrete. For cribs enclosing small areas the outer walls thus filled are all that are necessary. In large cribs, however, cross-partitions, similar to the enclosing walls, are constructed. One longitudinal partition will ordinarily be sufficient, but there may be several transverse partitions. This construction divides the enclosed space into several square or rectangular divisions, open top and bottom. See Figs. 41, 41 (*a*), and 42, (3) and (4).

3. A sufficient height of crib being built, either floating or on land and then launched, the crib is then floated and anchored over the proposed site of the pier and held in position by clusters

of piles, anchors, etc. The building is continued until the crib rests on the bed of the river or sinks some distance into it. Then the work of removing the material on the interior is commenced. Many more or less crude means of doing this have been practiced, such as ordinary scoops or iron buckets, connected to and worked by suitable gearing and machinery on top of the crib itself, or resting on platforms or barges. But now some form of clam-shell dredge is generally used. This may be defined as a large bucket, composed of several sections so hinged and connected that when it descends the sections separate, and its weight forces itself into the soil or around and over bowlders; when lifted, the segments close together on the material, which is then lifted to the surface and either emptied into the river or, when necessary, into barges. While the material is being dredged out the crib is built up, the pockets filled with the stone or concrete. With the relief from resistance on the interior and the weight of the structure, the crib sinks into the material at the bottom, either gradually and continuously or at intervals, depending on the resistance and weight. The method is simple and was formerly resorted to for great depths, either where it was not desired to drive piles or where the ordinary coffer dams were either unfit or too costly. For depths, say from 30 to 100 feet, the pneumatic process has, of late, been largely substituted; but the crib has been used where the amount of work would not justify the necessary first cost of the pneumatic plant or, for the same cause, it would be more expensive. Of late, however, in a few instances foundation beds at a greater depth than 100 feet below the surface have been required. As this depth is generally considered the limit of the pneumatic process, builders have resorted again to the use of the crib, either constructed of timber or iron, and to the iron cylinder. Three examples of this method will be briefly described.

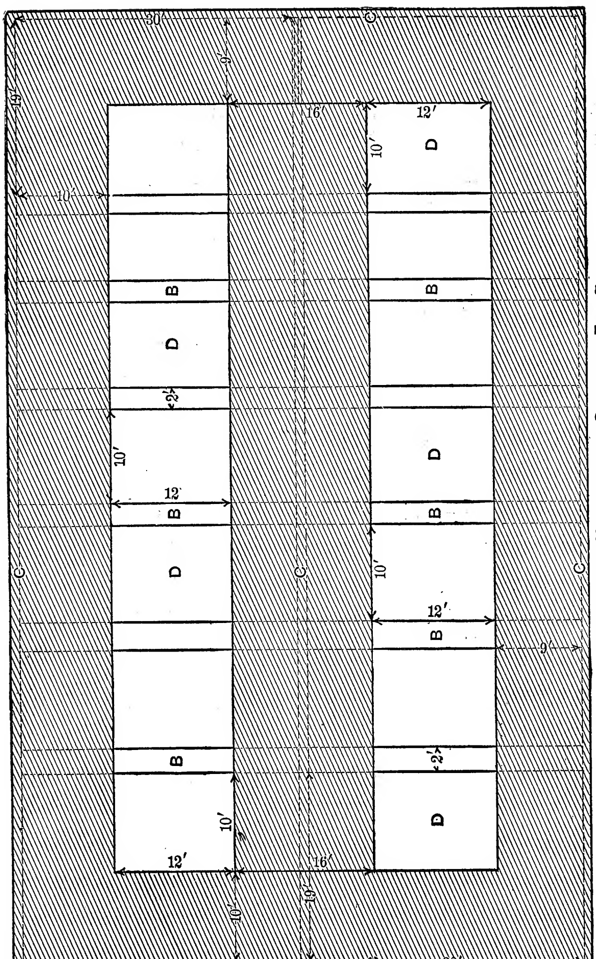
4. The design of a timber crib suitable for the above-described purposes is fully shown in Fig. 41. Fig. 41 (*a*) shows a plan or horizontal section; Fig. 41 shows a cross-section and part elevation; Fig. 42, (3) and (4), shows other details, etc.

This is a good example of the general construction of a crib, although it was in part designed for a combined crib and pneumatic caisson. It will be more fully explained farther on.

5. One of the longest and largest structures in which the open-crib method was used in the foundations is the Poughkeepsie Bridge across the Hudson River, New York, full descriptions and illustrations of which can be found in the *Engineering News* and the *Engineering and Mining Journal*. The following are the principal points of interest: There were two cantilever spans of 548 ft., and two counter balance or anchorage arms of 201 ft. each, one cantilever span 546 ft., and two contiguous through trusses of 525 ft.—giving a total length between end piers of 3094 ft., and including viaduct approaches 6767 ft. The grade on the approaches was 66 ft. per mile; clear height of structure above high-water 130 ft., making base of rails, as deck spans were used, 212 ft. above high-water. All masonry was of first class for facing stones, the backing being of concrete with large stones imbedded, so as to tie the face and backing thoroughly through the entire pier, as has been described under the head of masonry. The masonry rested on the cribs at about 10 ft. below high-water, and was built to about 30 ft. above high-water; on top of the masonry steel towers about 100 ft. high were erected, upon which the superstructure rested. To a depth of 100 ft. or more below high-water, the bed of the river was composed of silt, clay, and sand, underlaid by layers of a firm, coarse gravel, between which and the rock, which was about 140 ft. below high-water, there was found a bed of compact gravel, upon which the structure finally rested at a depth of about 135 ft. below low-water. There were 4 cribs of the same general design and dimensions. Bottom dimensions 60 × 100 ft., height 104 ft.; the dimensions decreased somewhat toward the top, giving a regular batter; they were built in the main of 12 × 12 in. hemlock, except the timbers which formed the cutting edges; these were of white oak. The lower section of the crib of about 20 ft. in height was built of the usual V-shaped section of solid timbers for the outside and cross-walls, similar to the lower part of crib

shown in Fig. 41. There was, however, only one cross-wall. The annexed diagram, Fig. 39, shows horizontal section at bottom of cutting edge (see dotted lines), and also at a point 20 ft. above, as seen by the full lines. *C, C, C* shows the cutting edges of the outside and middle walls; *B, B, B* cross-bulkheads 2 ft. thick dividing the enclosed space into 14 cells or pockets, open bottom and top and extending from bottom to top of crib. These are the dredging chambers or compartments. The width of the cutting edges was only a few inches, and these walls then increased in the height of 20 ft. to 10 ft. on the sides, 9 ft. on the ends, and 16 ft. in the middle walls; these are shown by the shaded rectangles. Upon these solid walls the double walls of the crib above was built which formed the cells or pockets for the concrete filling. It is seen that the dredging chambers *D, D, D,* are for the 4 end ones 19×30 ft. $= 570$ sq. ft. at the plane of the cutting edge, and the intermediate ones are 10×30 ft. $= 300$ sq. ft.; whereas at 20 ft. above in the plane of the shaded portions all chambers are 10×12 ft. $= 120$ sq. ft., and continue this size to the top of the crib. Such cribs are built either partly on shore and then launched or entirely while floating; when a sufficient height is built to reach from the bed of the river to a point somewhat above water surface they are floated into position and held by anchors, or clusters of piles, or by cribs loaded with stone and sunk at convenient points. The building of the walls of the crib, the weighting of the caisson with concrete, gravel, or broken stone is then proceeded with. The material is dredged from the bottom through the open chambers *D, D, D,* and as the material is removed and frictional resistance decreases, the crib settles into the soil. In this structure the weight supplied was gravel, and afterward this gravel was removed, as I understand the description, and then these same pockets filled with concrete, as was also the dredging chambers *D*. The settling of the caisson was somewhat uncertain and irregular, dropping sometimes as much as 10 ft. at once. This uncertain and irregular settling is one of the difficulties attending this open-crib method. Under ordinary cir-

OPEN TIMBER CRIB, POUGHKEEPSIE BRIDGE.



PLAN AND HORIZONTAL SECTION FIG. 7.

FIG. 39.—SHOWING CUTTING EDGE C, C, C, AND HORIZONTAL SECTION AT TOP OF V-SHAPED BOTTOM SECTION.

cumstances with the walls of the pockets well calked; there should be no difficulty in using concrete for the weight, and thereby saving the time and cost of first filling them with gravel or stone and subsequently removing the same. At least such is the writer's experience in cribs 40 or more feet in height, even though some of the pockets were often left unfilled to the depth of 15 or 20 ft. below the water surface. A sufficient margin on the height of the crib should always be provided to keep its top above water, and but little pumping should be necessary to keep the pockets free of water. Much of this concreting must have been done under water, which certainly is to be avoided if practicable. If such pockets had to be filled first with gravel or broken stone, which is then removed and replaced with concrete under water, it would have probably resulted in as good a job, if pipes a few inches in diameter with a series of holes at different levels had been built in the gravel or broken stones at intervals, and instead of removing the material, to have poured a grout made of cement alone, or at most with 1 cement and 1 sand, into these pipes, the head would force it through the holes and out between the gravel or stone, thereby more or less perfectly filling all interstices, and doubtless making as good a concrete as that ordinarily resulting from concreting under water. A somewhat similar plan has been tried, not on such an important and extensive work, perhaps, but is said to have given good results. The above described structure is specially noted for the size and height of the cribs and the depth of 135 ft. sunk below high-water. Although in many details the design and construction of these cribs may be different, yet the figures of Fig. 41 considered as a crib alone, without the shafts, pipes and horizontal partitions or roofs of the separate chambers, will represent a good design of all forms of open cribs; hence more elaborate drawings showing in details the cribs of the Poughkeepsie piers are omitted, and for these the reader is referred to the magazines mentioned.

6. Another bridge of great length and involving many difficulties, in which the open-crib method was used, was recently constructed by the Union Bridge Co., of New York, and known as the Hawkesbury Bridge, in New South Wales. In this case the cribs were constructed entirely of iron; the horizontal sections of the crib were rectangular with rounded ends, spreading out from a point about twenty feet above the bottom. Except in regard to the shape of the cribs, the number of dredging tubes or cylinders, and the thickness and the strength of the plates, angle-irons, etc., the elevation given for the crib of the Diamond Shoals Lighthouse, designed by Messrs. Anderson & Barr, will be ample without further drawings to represent this particular case. And as Fig. 41 has been taken as a fair type for the construction of all timber cribs, so may the figures in Fig. 40 be taken as a fair type of the all-iron cribs. Before giving some of the details of the Hawkesbury Bridge a few remarks on the general construction of iron cribs will not be out of place. By referring to Fig. 40 it will be seen that the lower section of the crib flares outward at a considerable angle; this has doubtless been characteristic of iron cribs, whereas in Fig. 41 the batter or outward flare is very slight, and the same may be seen in the plates showing pneumatic caissons. In either case the object is twofold. First, it increases the area of the base, thereby reducing the unit pressure on the foundation-bed; and, secondly, is supposed to facilitate the sinking of the caisson or crib by reducing the friction on the exposed surfaces. So far as the first consideration is concerned, the bottom could be made of the required area, this continued for a certain height, and the area reduced abruptly to the size required for the structure above; this, then, has no material importance. As to the batter facilitating the sinking it has generally been considered as absolutely necessary to have some batter; the amount, however, has been different in different designs. Mr. Anderson, who has had great experience in sinking deep cribs and cylinders, expresses the opinion that in running sand and silt it makes but little difference whether they have any batter or not; but if the material is tenacious, as in clay and compact silt, that a vertical surface on the outside of the lower section is to be preferred, as the material will not otherwise close in on the sides of the caisson, and that it would be more difficult to guide and hold the structure in a proper position. To confirm his view he states that both plans were tried in the Hawkesbury foundations, and all of the trouble occurred with the inclined sides, and little or none with those cribs that had vertical sides. The following table gives the depths sunk and total heights. The tops of the piers were 42 feet above low-water; difference between high and low water, about 5 feet.

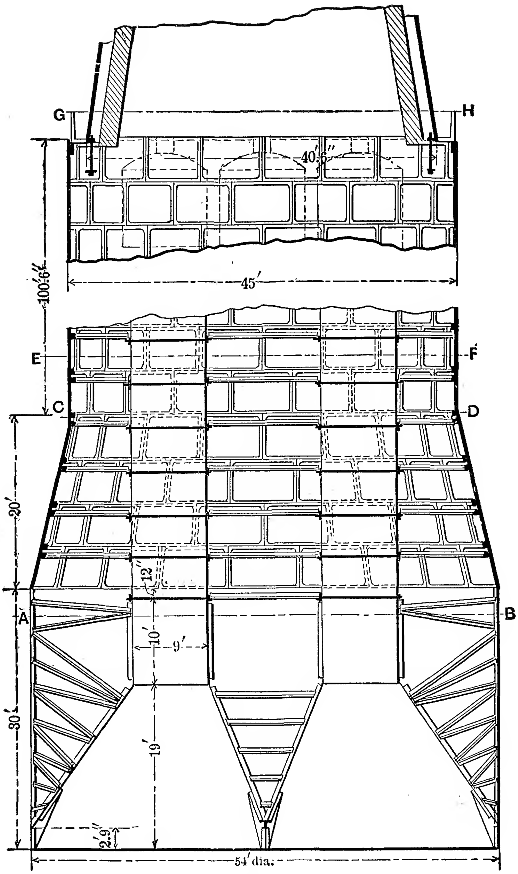
	Depth from Low-water to River Bed.	Depth Below River Bed.	Total Height from Bottom to Top of Pier.
Pier No. 1	38 ft.	55 ft. 8 in.	135 ft. 8 in.
" 2	40 "	108 " 1 "	190 " 1 "
" 3	43 "	96 " 0 "	181 " 0 "
" 4	21 "	118 " 6 "	181 " 6 "
" 5	19½ "	117 " 5 "	178 " 11 "
" 6	47 "	108 " 0 "	197 "

Some difficulties were encountered, as would have been anticipated in a structure of such magnitude.

The spans were constructed on false work erected on very large barges, and floated in between, and then lowered on the piers.

The length of this bridge was 2896 ft. in length. The depth to be sunk was as shown in the above table, through water, mud, and sand, finally resting on a bed of compact gravel. Such were the general dimensions, requirements, and results. In 1884 invitations were extended to the bridge builders in many parts of the world. The builders were to submit their own plans, both for the substructure and the superstructure, subject to certain limitations as to dimensions and strength of materials. A large number of plans were submitted by English, French, and American builders, which resulted in the contract being awarded to the Union Bridge Co., of New York, for the gross sum of about $1,835,000. No official or full particulars of this structure have been published by the builders. The following general facts, to which have been added some calculations of weights, resistances, etc., by the writer, are taken from the columns of the *Engineering News*. The total

FIG. 40.



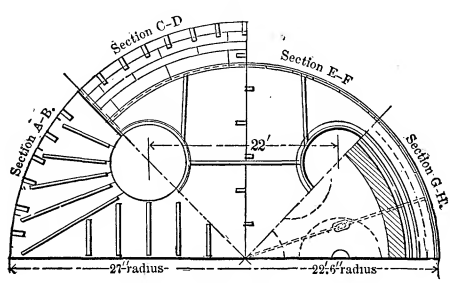


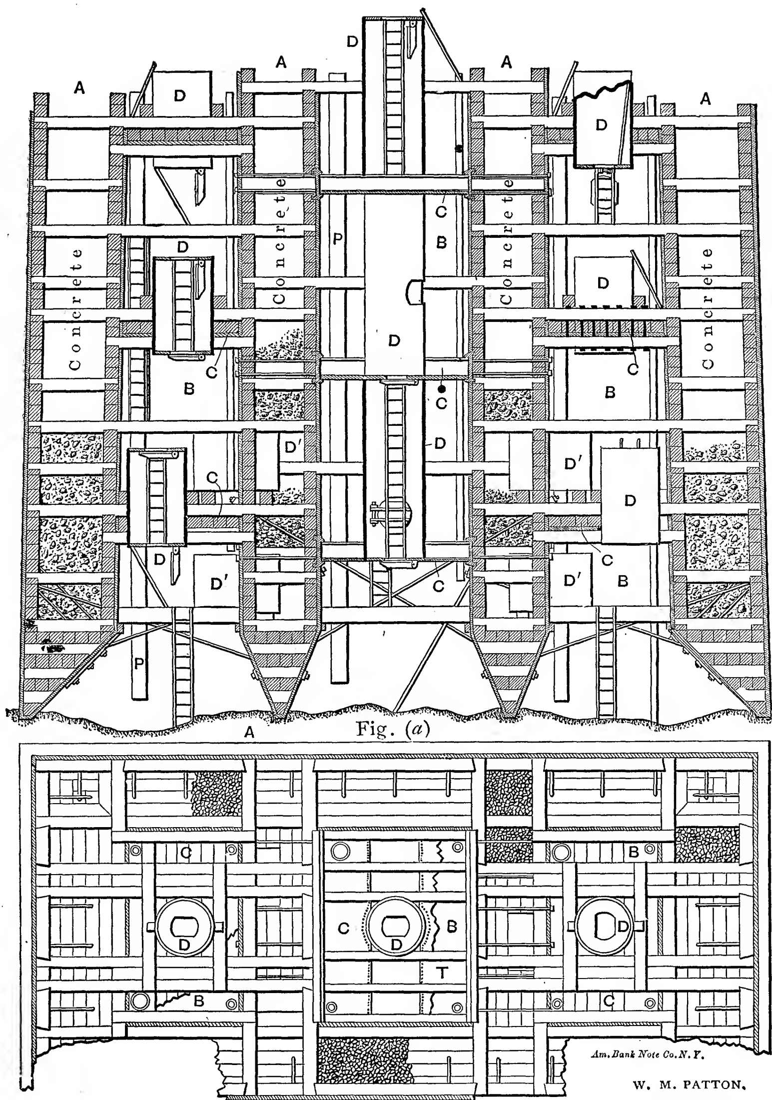
FIG. 40*a*.

CAISSON FOR DIAMOND SHOALS LIGHTHOUSE. DESIGNED BY ANDERSON AND BARR.

Fig. 40*a*, Horizontal Sections. Fig. 40, Vertical Section.

[*To face page* 269.]

FIG. 41.



COMBINED CRIB AND PNEUMATIC CAISSON.

Fig. 41, Vertical Longitudinal Section. Fig. (*a*), Horizontal Sections at Several Points.

[*To face page* 271.]

length was divided into five spans 416 ft. long each, and two spans each 408 ft. long, by six piers and two abutments. As the depth to be sunk far exceeded the generally accepted limit of the pneumatic process, it was determined to use the open-crib method. The crib was constructed entirely of iron. Except that the enclosing walls were composed of iron plates stiffened by angle-irons and strong iron braces between the double walls, the general design was the same as in timber cribs. The iron plates of the outside and partition walls were $\frac{3}{8}$ in. thick, the necessary weight to sink the crib being deposited between walls. These walls enclosed three tubes or cylinders 8 ft. in diameter; these extended to about 20 ft. from the bottom, at which point they commenced to swell out in a bell or funnel shaped mouth to the bottom edge, forming with the outside and partition walls strongly built and connected cutting edges. The horizontal sections were rectangular with rounded ends, the dimensions of the bottom section being 52×24 ft.; these dimensions gradually decreasing upward, so that at a point twenty feet from the bottom the cross-section was reduced to 48×20 ft., and thence continued at these dimensions for a height of about 155 ft. to low-water. This was built up in sections of about 5 ft. as the dredging and sinking progressed. The tubes were connected with the side and partition walls by strong iron braces. The entire open space around the tubes was filled with concrete as the sinking progressed; this, with the weight of iron, overcame the resistance. The material was dredged out through the tubes by means of the Anderson Automatic Dredge; each bucketful had to be lifted the full height of the crib at the time and deposited in the water or in barges. When the proper depth was reached the tubes were filled with concrete, deposited under water. On top of the crib masonry piers were constructed, about 40 ft. high; these piers were 42×14 ft. on top, and 46×18 ft. at the bottom, leaving a margin of about 1 ft. all around on top of the crib. The piers seem to have been constructed of two circular columns of masonry 14 ft. in diameter, and 28 ft. centres, connected by a rectangular wall 6 ft. thick at top, thereby saving some masonry.

7. The difficulties in this method of sinking such large structures are many. Great skill is required in handling the dredges so as to excavate the material uniformly and close up to the sides of the cutting edges at such great depths below the surface of the water; the importance of which, in sinking caissons and cribs, is very great, in order to maintain the structure in a vertical position and prevent careening and consequent sinking out of line and position. But the success attending such efforts fully establishes its practicability, though much is left to blind chance. The sinking must, to a great extent, take care of itself. Again, if obstacles, such as old wrecks, drift, logs, etc., are met with, the removal of these causes great trouble and delay with its attending cost, as it is by no means an easy job to remove such obstructions in the pneumatie caisson, where they can be seen and reached. Much of the concrete is of necessity deposited under water, the value of which was fully discussed under the head of concrete. If deposited with care, the operation is slow and expensive, and without care it is no better than so much broken stone, and perhaps not much better with any degree of care.

Lastly, the frictional resistance of the material on the exterior surface of the caisson is enormous, especially if the sinking is intermittent, allowing intervals of rest, during which the material closes in on the caisson. This requires corresponding and enormous weight to overcome it. This resistance may be many hundred pounds per square foot of surface. As an illustration, the writer has made the following calculations on this structure: Allowing the low unit of resistance of 250 lbs. per square foot, the total resistance must have been 12,000 × 250 = 3,000,000 lbs. The weight of iron, roughly estimated, would be 550,000 lbs., leaving 2,450,000 lbs. of concrete to be added, which, at 125 lbs. per cubic foot, would require 20,000 cubic feet, and allowing a reasonable excess, say 1000 cubic yards. Again, the estimated total weight on the foundation bed would be 98,806 × 125 = 12,351,150 lbs. of concrete; weight of iron 550,000 lbs.; masonry, 15,300 × 160 = 2,448,000 lbs.; superstructure and load, 2,113,280 lbs, or a total of 17,-

FIG. 3. FIG. 4.

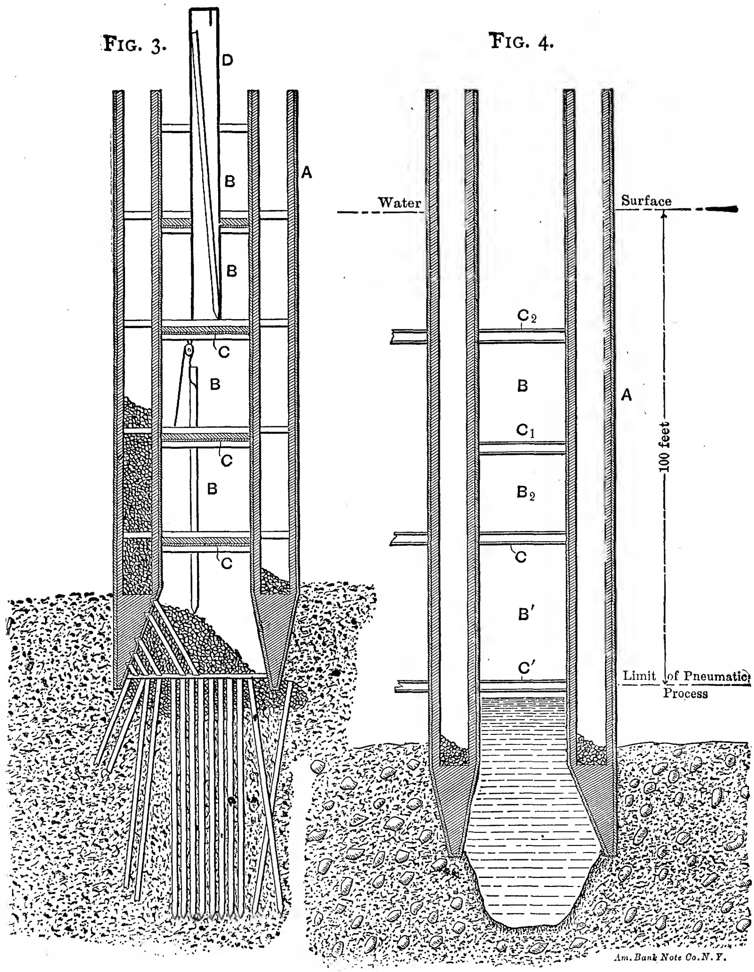


FIG. 42.—COMBINED CRIB AND PNEUMATIC CAISSON.

Fig. 3, Part Vertical Section. Piles driven in Interior. Fig. 4, Part Vertical Section, Sunk below Pneumatic Limit.

[*To face page* 272.]

462,430 lbs., or 13,992 lbs. = 7.0 tons per square foot, not considering the frictional resistance, or 5.8 tons, allowing for it. The writer regrets his inability to give fuller information on this structure.

8. In 1885 a prominent bridge builder consulted with the writer in regard to the cheapest and best method of reaching such a depth, as the pneumatic process was considered out of the question, and it was feared that the open-crib method would prove impracticable on account of the many difficulties and objections already mentioned. Being so fully occupied at that time, he could not give the necessary consideration to the matter. But in the following year he designed a structure which was intended to be a combination of the open crib and pneumatic caisson, involving some new features which were subsequently patented. This will be more fully explained in another article, after explaining the pneumatic process.

9. The third example of the open-crib method will be briefly alluded to. It was required to construct a bridge across a wide, deep bayou at Morgan City, Louisiana. The material of the bed of the stream was very soft, with considerable depth of water over it. Several plans had been discussed and submitted while the writer was connected with the road. Among them was the Cushing cylinder piers, and timber piles with cast-iron cylinders connecting with them at or near the bed of the river, as well as others of more or less cost. But the work being abandoned, nothing had been done beyond driving a few piles. Subsequently, on the renewal of the work, it was determined to sink iron cylinders by means of dredging out the material. These cylinders were eight feet in diameter. Below the bed of the river they were made of cast-iron in sections 10 ft. long, with $1\frac{1}{2}$-in. metal thickness, strongly bolted together through internal flanges. Above the bed of the river wrought-iron plate, $\frac{3}{8}$ in. thick was used, riveted together and stiffened with angle irons. The material was dredged out from the interior of the cylinder, and as the cylinder settled sections were built on top. By these means they were sunk a hundred or more feet into the solid material. After reaching the proper depth, they were filled with concrete. The stability of such small columns, having long distances unsupported, has been repeatedly noticed. They certainly cannot be regarded as possessing any great excess of stability where they are subjected to heavy pressures or great shocks, especially as the concrete has to be generally deposited under water. The above examples illustrate the most recent open-crib and cylinder constructions in which depths have been reached exceeding that to which the pneumatic process is generally considered applicable, which will now be explained.

ARTICLE XLVIII.

THE PNEUMATIC CAISSON.

10. BEFORE describing the designs and construction of caissons, it will be as well, to avoid repetition, to briefly consider certain general principles applicable in all cases, and also the design and uses of certain parts common to all.

11. As the name indicates, the air is an essential element to be considered, whether simply used to sink the caisson, 1st, where a vacuum is made by exhausting the air from the interior of an air-tight cylinder or box, and the unbalanced atmospheric pressure of 15 lbs. per square inch of exposed surface, causing it to sink into the underlying material. This is called the vacuum process: it may be said that it is rarely, if ever, used now; 2d, where the air is compressed into a cylinder or box, which drives the water out, so that the material can be excavated and removed from the interior, which is called the air or working chamber, lifting it out in buckets, or allowing the air to blow the material out through pipes properly regulated by valves, or forcing it out by water pressure. This is known as the compressed-air or pneumatic process. This latter term is now commonly confined to the use of compressed air.

12. The fundamental principle underlying this is simply

that the atmospheric pressure of 15 lbs. per square inch will support a column of water, in a tube or pipe from which the air has been exhausted, of about 34 ft. high, when the open end is immersed in a body of water; or 1 lb. will balance a column of 27 ins. high. Practically these heights cannot be supported, as a perfect vacuum is almost impossible. But it is commonly stated that we must have 1 lb. pressure for every $2\frac{1}{4}$ ft. of depth below the water surface, to keep the water out of the working chamber. The actual pressure is 15 lbs. more, as we have to balance a like pressure on the surface of the water outside the caisson; this excess is constant for all depths. So that if the depth below the water surface is 90 ft. the actual air pressure in the caisson is about $45 + 15 = 60$ lbs. The uplifting effect is, however, only 45 lbs. Ordinarily, it becomes necessary to reduce the air pressure in the caisson very materially at times in order to allow the caisson to sink; at other times, however, it is necessary to cease altogether adding weight to the caisson to prevent a continuous or too rapid sinking. This, of course, depends both upon the actual resistance at the lower or cutting edge of the caisson, which may or may not be very great, and upon the frictional resistance on the exterior surface of the caisson and the structure upon it. It is therefore, in general, better to have as little frictional resistance on the side surfaces as practicable, and to provide as great a direct resistance at or a little above the cutting edge as is consistent with economy and convenience of construction and subsequent ease of prosecuting the work.

13. As the working chamber should be practically air-tight, some special means of entering and leaving the working chamber must be provided. The air-lock has this object in view, and wherever it is placed or whatever its design, it must be an air-tight box with two doors, both opened toward the greatest pressure—that is, toward the air-chamber or some air-tight channel or shaft communicating with it. These doors open inward or downward, and when shut must bear against rubber gaskets, so as to practically exclude the passage of air; as it is the air-pressure itself that keeps the door shut, one of

them will always be open. Strong and tight iron shafts are built into the caisson, and should always reach well above the surface of the water; the main shaft through which the men enter and leave need not be over 4 ft. in diameter. This is made in sections, which are bolted together through internal flanges, between which rubber bands or some soft and impervious substance is placed, so as to render the joint air-tight. Ordinarily red lead worked up with short strands of ordinary lampwick will answer every purpose, it is easily obtained and applied. A section of the shaft itself can be converted into an air-lock by connecting two doors to it, or a specially designed air-lock can be connected with the shaft at its top, bottom, or any intermediate point. The writer prefers the air-lock at the top, and that it shall also be simply a section of the shaft; as any section can be converted into an air-lock, or the whole shaft if so desired. This arrangement possesses many conveniences, and is much safer than when located at or near the bottom. It frequently happens that men are driven suddenly from the working chamber, and if the lock is at the top they can all climb up the shaft and be in safety, while the air is being equalized so that the lower door of the air-lock can be opened, or if open they can enter the air-lock without delay or confusion, or the danger of some one closing the door upon them. On the contrary, with the air-lock at or near the bottom, the men have no place to enter and be safe if the lower door of the lock is closed; a few minutes' delay may be fatal to many, or they all may not be able to enter the air-lock in the confusion and often cowardice shown by some men in the face of danger. The air-lock being a part of the shaft is a mere matter of convenience.

14. A smaller shaft, not over 18 ins. or 2 ft. diameter, for letting concrete or other material into the working chamber, is also used. It is better to have at least two of these; they are provided with a door at top and bottom only, the entire shaft being an air-lock. In addition to these, pipes from 4 to 6 ins. diameter are also built into the caisson—the larger diameter for connection with the air-hose and force pump for water, the

smaller diameter for use in blowing out the material. There should be a number of these distributed around the caissons. All pipes should be provided with the best valves, and when not in use should be capped with a cap screwed on to the pipe above the surface and stopped by plugs below to prevent any possible chance of a sudden escape of the compressed air.

15. The use of the air-lock can now be easily understood. Compressed air is rarely, if ever, required until the caisson rests firmly on the bed of the river in its proper position for the pier. As soon as it does so rest, the doors being both open, air connections are made between the proper pipe and the air compressors; all other pipes or avenues through which the air could escape being closed, the lower door is lifted by a small tackle against its bearing, and the compressors are then started. It requires only a few pounds of pressure to hold the door in position. When the pressure gauge indicates a pressure required for the then depth, men enter the air-lock through the other doorway, its door swinging freely. This door is then lifted into position by the lock tender on the outside; the valve in the upper door or in any other position in which it may be placed is closed, and the valve in the lower door or opening into the main shaft at some point below the air-lock is opened. The compressed air rushes into the air-lock, and continues to do so at a lessening velocity until the air in the lock is at the same pressure as that in the working chamber; it is then said to be equalized. The lower door would now open of its own weight, if it were not held in position by a tackle in the air-lock. As the pressure on both sides is now the same, the lock tender on the inside allows the door to open, and the men descend by means of an iron ladder fastened to the sides of the shaft into the working chamber. A thorough examination is made to see that there are no leaks; complete the interior bracing if not already completed, and see in short that everything in the interior is all right. To get out they ascend the shaft, enter the lock through the lower or open door-way, lift this door to its place, close the lower valve, and open the upper valve, which allows the compressed air in the lock to escape into the open air.

In a short time this pressure will be reduced to that of the atmosphere, the upper door is lowered by the outside lock tender, and the men pass out. The above operations have to be repeated each time that a man passes in or out of the caisson.

16. If everything is ready below, a gang or shift of men now passes into the lock and thence into the caisson, and the work of excavating the material in the caisson is commenced. So long as the depth is not over from 60 to 70 ft. below the water surface, only three gangs or shifts are required during the 24 hours; each shift working 8 hours and resting 16 hours, coming out to lunch at about the middle period of their working time. This will consume from $\frac{1}{2}$ to $\frac{3}{4}$ hours, so that they only remain about $3\frac{1}{2}$ hours in the caisson at a time. For greater depths the men are divided into 4 shifts, working 6 hours each, with the same interval of rest during this time, or actually remaining in the caisson about $2\frac{1}{2}$ hours at a time. A full shift consists of 1 foreman and 10 to 20 men, according to the size of the caisson, and one outside and one inside lock tender; this not including the machinery men, such as engineers, firemen, pipe-fitters, etc., and one or two handy men, and over all a general superintendent. The general duties of these men and the mode of procedure will be explained later.

17. One thing can be relied on: so long as the air pressure required by the depth is maintained, the water will not rise above the extreme lowest line of the cutting edge of the caisson, and in sinking through some materials water has to be pumped into the caisson in order to carry on the work. The caisson must be heavily weighted before the air pressure is put on, or a dangerous tendency to lift and careen will exist. The end of the air pipe in the caisson should be fitted with an automatic valve, opening into the caisson, so that should the compressors stop from any cause, the air pressure will close the valve and prevent the escape of the air; a simple circular plate of iron with a rubber gasket sliding freely on two small iron rods attached to the end of the pipe, and allowing a play of $1\frac{1}{2}$ to 2 ins., answers well the purpose, as it does not prevent an easy flow of air into the caisson, but closes instantly on the air compressors stopping. A small plunger pump connected with the compressors forces a certain amount of water in with the air to prevent its getting too dry and hot; this is all important. At a depth of 80 or 90 ft. the usual temperature in the working chamber will be from 85 to 90° Fahr. This is due to compressing the air. The temperature of the air in the air-lock will rise to 106 to 125° Fahr., the temperature in the air chamber being reduced by the moisture and the cooler surfaces on the interior.

18. As to the effect on men working in compressed air, a few remarks may be interesting and instructive.

While in the air-lock everybody is more or less affected with pains in the ears, known as "blocking." With some it is intense, and many have to reverse the valves and get out before the pressure is equalized, but the act of swallowing, blowing the nose, or closing the nose and mouth and exhaling the air from the lungs will give ready relief. This trouble may arise either on entering or leaving the lock. Again, in about 15 or 20 minutes after coming out of the caisson many men are attacked with severe pains in the limbs; these may be more or less intense and may last a day, a week, and sometimes longer, but seem to leave no permanent effects. These pains are known as the "bends." Returning into the compressed air gives instant relief, but they will probably return on again leaving; this trouble is common, but very many escape entirely.

A more serious trouble sometimes happens, resulting in a paralysis of some part of the body. This will in general be of a temporary nature, but is sometimes lingering and often permanent; but a small per cent of men will be thus attacked. And lastly, some severe cases of paralysis occur, from which the men die within a few hours or in a day or two. Occasionally a blood vessel in the nose or ear will be broken, some men losing their hearing from this cause; on the contrary, for some forms of deafness it has been claimed that exposure to compressed air affords more or less relief. Many opinions and theories have been advanced as to the principal causes of these troubles; but most, if not all, are unsatisfactory, if not entirely erroneous. The writer had about five years of almost continuous experience in works of this kind, going almost daily into the caissons and remaining often in the pressure for hours at a time; and though not conscious of any harmful effects of either a temporary or permanent character so far as he was concerned, he made a careful study of the effects on others, and believes that much of the trouble is due to carelessness and indifference on the part both of the men and managers, and even under these circumstances he believes that perfectly healthy men have but little cause of uneasiness.

19. In going through the air-lock, as has been stated, the temperature rises in a few minutes from that of the atmosphere at the time, whether during freezing or milder weather, to at least 106° Fahr., causing a profuse perspiration to set up in a few minutes. This continues while below. On passing out through the air-lock, as the pressure rapidly falls, so does the temperature; the perspiration is suddenly checked, and a cold, clammy sensation follows. The men, with little or no clothing on, pass out into a temperature very much lower, often well below the freezing point; they sit in exposed positions around the engine room or elsewhere for a half hour or more, and again go through the same ordeal. Entirely inadequate arrangements for their protection or comfort are sometimes provided. Entering and leaving the caisson often happens many times in a period of six or eight hours. While working below, even if the working chamber is lighted by electricity, which is not always or even generally the case, it is necessary to use candles to a great extent; these are especially prepared, and would burn but slowly under ordinary conditions, yet burn freely in the compressed air, saturating the air with large quantities of soot, which the men breathe freely and constantly, getting their system and lungs filled with it, and expectorating continually a black mass from their lungs—this continuing for weeks even after completing the work. The above conditions are doubtless the most potent factors in causing the caisson diseases. It is commonly believed that the actual pressure is the cause. There is absolutely no evidence to sustain this opinion, beyond the blocking of the ears, which is evidently caused by an almost infinitely small period in time of an unbalanced pressure on the outside or inside of the drum of the ear, as in all other respects the condition of the physical man is perfectly normal, no matter how long he may remain under the pressure. There is no observable compression or subsequent puffing of the flesh, no restraint or other change in his movements, or in the use of himself, except that he will work and hit harder and feel more or less exhilaration, which is no doubt due to an increase in the supply of oxygen, which even overcomes the lassitude that would otherwise be caused by such a profuse and continued perspiration.

20. If the writer is right in his views, the remedy or certainly an amelioration of the troubles is simple and not expensive.

1. Select only healthy men for this work. Little or no attention is given to this. The only rule is to get men and get them as cheaply as possible. From 20 to 25 cents an hour for eight hours' work is the usual price paid. Men will do from one and a half to two times the work in the caisson that they will do outside.

2. Prevent, at least, to some extent, the sudden alterations of temperature through 70 to 90° Fahr. day or night, and in all kinds of weather. A common reply to this is that the men who regulate the valves are instructed to pass the men through slowly, and that the valves are worked entirely by the men in the locks, who will be the sufferers; but yet valves of comparatively large apertures are given them. If they were smaller, or if not fully opened, the time would be much longer in passing in and out, during which time all work must be suspended in part or entirely; this means loss of time and money, which is not necessary. Provide then a lock or chamber connected with the main lock, in which men can enter without obstructing the main lock, which can be maintained at a bearable temperature, while the air is being equalized; let the men wash and dress themselves, and come out in some sort of comfort. Any man will get out of a temperature of 106° to 125° as fast as

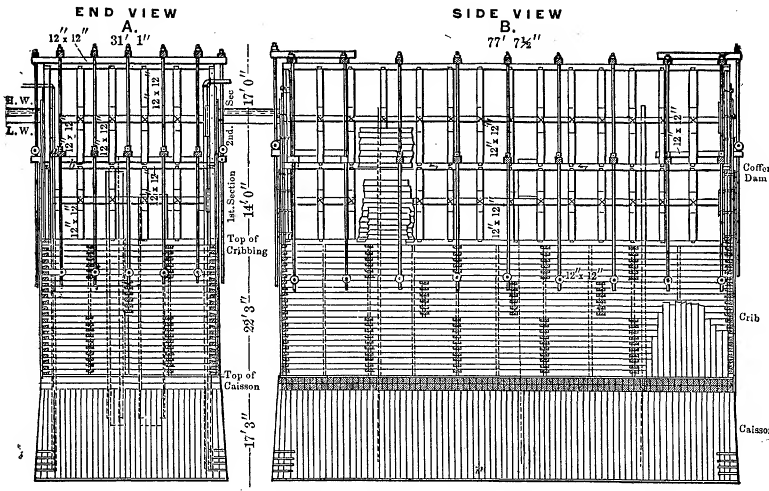
possible; nor will he remain in a cold, clammy condition longer than possible. Although it may not be practicable to do away entirely with candles, the use of them can be materially lessened. These remedies will be attended with some expense, but they will greatly add to the health and usefulness of the men, and doubtless enable us to reach much greater depths than 100 ft. by the pneumatic process with vastly less danger and suffering than now exists at depths under 100 ft. below the water surface.

21. A code of signals is always used, by which the men in the caisson can communicate their wants to those above. The method of simply knocking with an iron bolt on the iron shaft or pipes is as satisfactory as any that could be devised; it gives a clear, ringing, unmistakable sound. 1 knock for more air, 2 for less, 3 for starting the water-pump, 4 that the men are coming out, etc., varied as may be desired, answers all practicable purposes. The outside lock tender above all should be a faithful, wakeful, and reliable man, ever on the alert for signals from below, as all wants should be supplied immediately.

22. The immediate effect of reducing the air pressure even by only a few pounds is to set up a dense fog. All oscillations in the pressure should therefore be avoided as far as practicable, and this together with the greater tax upon the capacity of the compressors is the main objection to forcing out the material through the pipes by means of the compressed air; a method which in other respects is more rapid, and in many cases more economical and satisfactory, than any other of removing the material from the working chamber. A 4-in. pipe will easily carry gravel, sand, mud, and bowlders up to $3\frac{1}{2}$ ins. diameter. It requires careful regulation or feeding, however, to avoid choking the pipes, and requires a considerable quantity of surplus air. For these reasons a sand or mud pump is often or commonly used.

23. A few remarks on the necessary machinery will be useful. Several boilers of large steam-producing capacity are essential; much time and money are lost and great inconvenience

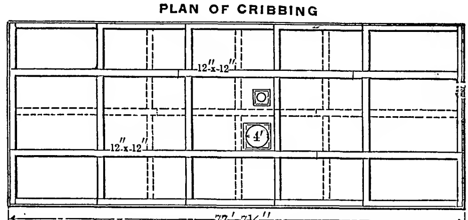
Fig. (*a*)

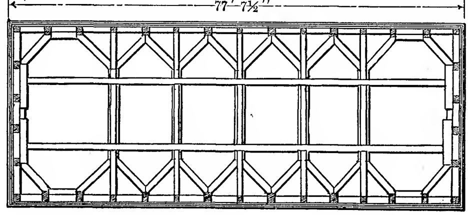


Total Timber in Caisson	316,689,50 Ft.B.M.	Displacement sinking Caisson	156,183,07 cub.ft.	
" " " Crib	143,993,14 "	" excavation below cutting edge	3,405,24 "	
" " " Coffer-Dam	108,517,61 "		159,588,31 "	
	569,200,25 "			

Fig. (*b*)

PLANS
C.
PLAN OF CRIBBING





PLAN OF COFFER-DAM AND SYSTEM OF BRACING

Fig. (*c*)

FIG. 43.—PNEUMATIC CAISSON, CRIB AND COFFER-DAM, SUSQUEHANNA RIVER BRIDGE, B. & O. RY.

[*To face page* 283.]

caused by the want of them. The compressors have to be run continuously day and night, and often in addition large force pumps, electrical machinery, and pumps for keeping water out of the cribs while concreting and out of the coffer-dams while building the masonry. And after making a liberal allowance for these purposes, at least one extra boiler should be provided, as some wear out, some need repairs, and a largely increased supply of steam is sometimes required. One good-sized double compressor will generally supply the requisite amount of air; another should always be in reserve. At least one large double force pump should be provided. Other engines, pumps, etc., of smaller power will be required. A large supply of pipes, hose, machinist tools, etc., should be provided, and with them a first-class machinist, as a large amount of fitting, repairing, etc., must be done on the work and promptly, whether required by day or by night. This machinery is generally mounted on one or more barges and tied to the structure. All connections between the machinery and the pipes, etc., should be made by the best make of hose, to avoid any possibility of breaking, bending, or otherwise deranging any of the pipes. As a sudden escape of air may cause not only loss of life, but serious damage to the structure. No worn-out, broken-down machinery or fittings of any kind should be allowed.

ARTICLE XLIX.

CONSTRUCTION OF PNEUMATIC CAISSONS.

24. THE general design of caissons is the same whether made of wood or iron, and consists of three parts, as follows: 1st, the walls of the working chamber; 2d, the deck or roof of the caisson, with its necessary shafts, pipes, etc., built into and through it; and 3d, the necessary trusses, braces, etc., to strengthen and stiffen the walls and the roof.

A short description of the design and construction of some of the typical timber and iron caissons heretofore used will now be given.

25. The caissons for the foundations of the New York and Brooklyn suspension bridge are about the largest timber caissons constructed in this country. They were rectangular in cross-section. The bottom dimensions 172 × 102 ft., and at top 165 × 95 ft., 31½ ft. high; thickness of the roof 22 ft., and were sunk 78 ft. below mean high-tide. The frictional resistance on the sides varied from 280 to 600 lbs. per square foot. Estimated pressure on a foundation-bed of sand, 7½ tons per square foot. The design of the caisson was simple. All of the timbers composing it were 12 × 12 ins. in cross-section and laid horizontally and well bolted together. The height of the working chamber was 9½ ft.; the thickness of the walls varied from 6 ins. at the cutting edge to 9 ft. where it was joined to the roof. This was built solid of timbers laid in courses one on top of the other, crossing each other and bolted together. The inner slope of these walls were 1 to 1, the vertical section being V-shaped. These were connected by cross-wall, also built solid, dividing the working chamber into compartments communicating by openings in the cross-walls. On these walls a solid roof of square timbers in courses crossing each other, 22 ft. thick, was constructed, thoroughly bolted together. A cast-iron shoe was placed on the cutting edge, and under this plate-iron was bent extending up both the outer and inner slopes of the wall, and in one of the caissons the entire inner surfaces of the chamber was lined with plate-iron, and also between the fourth and fifth courses of the roof a layer of tin was placed and bent downward on the outside, reaching to the iron plate above mentioned. These metal linings were used to prevent damage from fire, and also to insure air-tightness. The deck timbers were not placed in close contact, the intervals being filled with concrete or mortar. The usual pipes, shafts, etc., were built through the roof, and in addition a large shaft 8 ft. in diameter, open at both ends, the lower end reaching into an excavation at the bottom filled with water, the water extending up the shaft. This was used for removing large bowlders, etc., by means of specially designed hooks or buckets worked from above. This was about the only novel or unusual feature in the design.

One of the caissons caught fire, which, being supported by a large quantity of oxygen, burnt its way to a considerable distance into the roof. The caisson had to be flooded to extinguish the fire. It is not an unusual habit of caisson men to use the flame of a candle to detect leaks in the caisson. There is always some danger in the presence of so much oxygen and combustible material of starting a fire. Other methods of determining air-leaks should be used.

26. The caissons for the St. Louis bridge, though commonly called iron caissons, were largely constructed of timber and iron combined. The walls of the working chamber were composed of iron plates, stiffened by angles and brackets; timber also being fastened to the walls, giving stiffness and also affording an increased bearing surface. The decks of these caissons were formed by deep and strong girders or beams, resting on the outside, and cross-walls of the air chamber, to the under side of which plate-iron was riveted or bolted, forming a strong and air-tight roof. The space between the girders was filled with concrete or masonry, and the regular masonry for the piers was then built on top of this. As the sinking progressed, a timber coffer-dam, sheathed on the outside with plate-iron, was built up, in which the masonry was constructed. In the Brooklyn bridge no coffer-dam was used; the masonry commenced on the deck of the caisson, and was built up as the caisson settled, so as to keep its upper surface above the water-line. In the St. Louis bridge large open shafts were built in the masonry; these were lined with brick and timber, so as to make it water-tight. The airlock was placed at the bottom of the shaft. The writer has heard it stated that in this, as in some other cases, the engineers placed the air-locks at the bottom, leaving long open shafts, reaching above the surface of the water, so that the men might ascend the ladders or the shafts in the ordinary air. Whether this is true or not, he does not think that caisson men would hesitate to prefer to make the ascent in compressed air, as there is always a feeling of lassitude and an indisposition to exerting one's self immediately after coming out of compressed air, to say nothing of the feeling of safety when the air-lock is at the top of the shaft. The horizontal cross-sections of the St. Louis caissons were hexagonal in shape to conform approximately to the shape of the masonry piers; their dimensions at the bottom were 83×70 ft., and 64×48 ft. at a point 14 ft. above. A section of this kind is easily built in iron, but for timber caissons it would present some objectionable features. These caissons, after sinking through water and sand, finally rested on rock at a depth of $109\frac{1}{2}$ ft. below the water surface. The sand was removed during the sinking by the sand pump, the principle of which is the same as the ordinary injector, and will be explained under another example of caissons. The working chamber of one of the caissons was filled entirely with concrete. But, as a matter of economy in the other, a wall of concrete was built entirely around the working chamber, and the interior space was filled with sand. The estimated pressure on the foundation-bed was 19 tons per square foot. At the time of constructing this bridge the caissons were the largest ever used, and the depth below the water surface the greatest ever reached. All things considered, this bridge is one of the greatest structures in the country.

27. The latest, and perhaps the largest, structure of the kind has recently been completed across the Mississippi River at Memphis, Tennessee. This bridge was opened for traffic May 12, 1892.

Although no full and official publication has been made in regard to this structure, the following data and description have been obtained from reliable sources.

The total length of the structure is 7997 ft., divided as follows:

Iron viaduct approach to bridge proper	2300.00 ft.
Timber trestle " " " "	3100.00 "
Bridge proper, divided into 5 spans by 6 piers. The length of the spans were as follows: 1 span 225.83 =	225.83 "
1 cantivever span. Cantilever arms each 169.38 ft.; suspended truss, 451.66 ft. Total length	790.42 "
1 span cantilever arm 169.38 ft., and truss 451.66 =	621.04 "
1 through truss	621.06 "
1 deck "	338.75 "
Total length	7997.10 ft.

The masonry piers varied in height from 93 to 158 ft., constructed of Georgia granite-face stones and Indiana limestone backing.

There were five pneumatic caissons, varying in horizontal dimensions from 40 × 22 ft. to 92 × 47 ft., and in height from 40 to 80 ft. from the bottom of caisson to bottom of masonry,* and sunk to depths from 78 to 131 ft. below high-water As it was apprehended that a scouring action might be caused by the obstruction to the current when the caisson was lowered to the bed of the river, rendering it difficult to properly level and locate the caisson in the commencement of the work, large willow mattresses, laced with wire, were constructed and sunk to the bed of the river over the site of the caisson by sufficient weight of rock. These mattresses were 240 × 400 ft. square, this affording a large, protected surface on the bed of the river. Upon these the caissons were lowered; and when they rested firmly and the air-pressure put on, men descended into the working chamber and cut through the mattress along the cutting-edge of the caisson, allowing the caisson to sink through the mattress. Total distance from top of masonry to foundation-bed, which was composed of clay, was about 199 ft., and below high-water about 131 ft., and 96 ft. below low-water. The greatest immersion was 108 ft.

The anchor pier on land was founded about 50 ft. below the surface, and weighed 2500 tons. Long iron rods passed through the masonry, and were fastened to a network of iron I-beams under the masonry. This made the entire mass of the pier act as a unit in balancing the moving load on the cantilever. The cost of this structure was about $3,000,000, and required about three years in its construction. The under side of the trusses were 109 ft. above low-water and 75 ft. above high-water. Height of trusses, about 78 ft., and width between trusses, 30 ft. between pin centres. The notable features of this structure were the great length of spans used,

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* The designers of these structures seem to make no distinction between the caisson proper and the crib above it; calling the entire structure a caisson. The writer calls that part a caisson shown in Fig. 51, (*a*), (*b*), and (*c*). A crib above may or may not be used.

especially in the cantilevers, and the use of large mattresses to prevent scouring action in the early stages of the work. Total weight of superstructure, 19,541,700 lbs., or 9771 tons.

The above are examples of the largest structures now in existence in which pneumatic caissons of wood alone or wood and iron combined were used in the construction of the foundations.

28. A description of an all-iron caisson will be given, accompanied with a skeleton sketch of the caisson itself—interesting and instructive not only as a typical iron caisson, but also from the special difficulties in the way of its completion, as well as in its entire failure. It was a Government work, and the object was to construct a lighthouse off the coast of North Carolina, on what is known as Diamond Shoals. The Government engineers advertised for plans and estimates of cost, leaving the matter of design and methods to be pursued to those desiring to bid on the work. The result was that three proposals were offered by American builders, differing somewhat in plans, cost, and methods of procedure. Owing to the exposed location of the structure and the severe and sudden storms, with the consequent excessive scouring of the shifting sands, the greatest difficulty necessarily arose in the commencement; and the success of the enterprise depended mainly on choosing the most favorable time and securing a good hold below the surface between the periods of the prevailing storms.

One of the plans called for a large timber caisson surmounted by a strong double-walled iron crib constructed on top of it; the spaces between the walls of the crib to be filled with concrete, in order to furnish necessary weight to sink the caisson.

Another plan in which both caisson and crib were to be constructed principally of timber and concrete used in the crib to sink the caisson. In both cases the working chambers of the caisson were to be filled ultimately with concrete.

The third plan, which was accepted by the engineers, provided for executing the work by the open-crib process, with alternate proposition for pneumatic caisson, if found necessary. This can better be described in the words of the contracting parties, Messrs. Anderson & Barr.

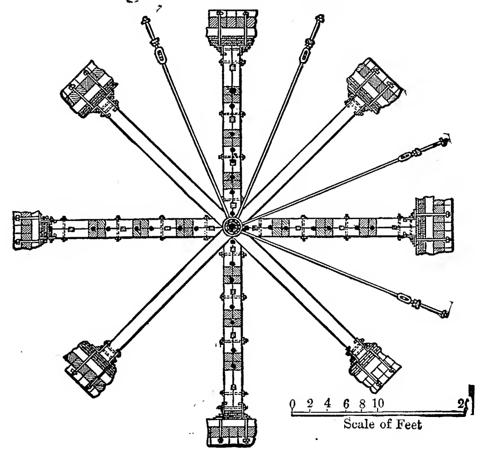


FIG. 46.

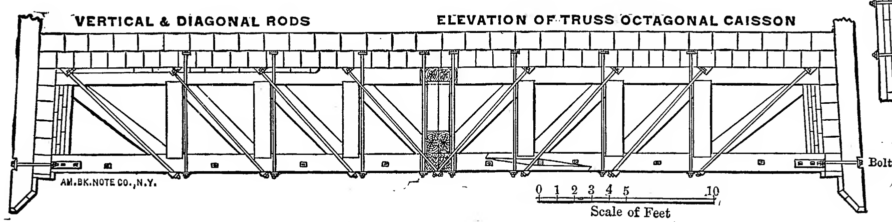


FIG. 45.

QUANTITIES:

Square Timber	217,845.35 Ft. B. M.	Concrete 707.23 Cub. Yds.
Plank	20,462.73 " " "	Excavation 116,084.96 Cub. Feet.
	Total Iron 58,618 lbs.	

DIAGRAM SHOWING DISTRIBUTION OF SCREW BOLTS AND DRIFT BOLTS IN VERTICALS OF OCTAGONAL AND RECTANGULAR CAISSONS.

2 Screw Bolts 41 ins.			3 Screw Bolts 41 ins.		
1 " " 36 "			1 " " 36 "		
1 " " 28⅝ "			1 " " 28⅝ "		
5 Drift " 30 "			5 Drift " 30 "		

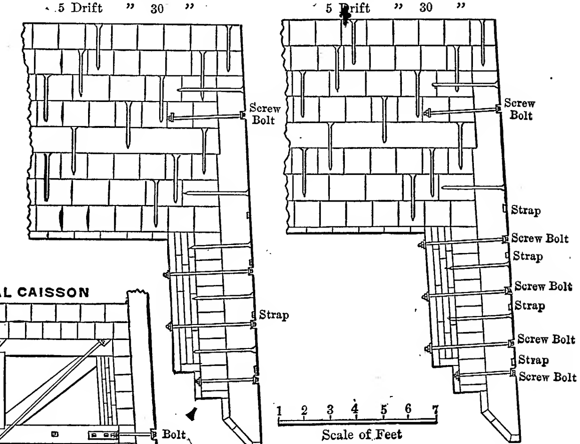


FIG. 44.

DETAILS OF CONSTRUCTION, OCTAGONAL CAISSON.

[To face page 288.]

Diamond Shoals Lighthouse.

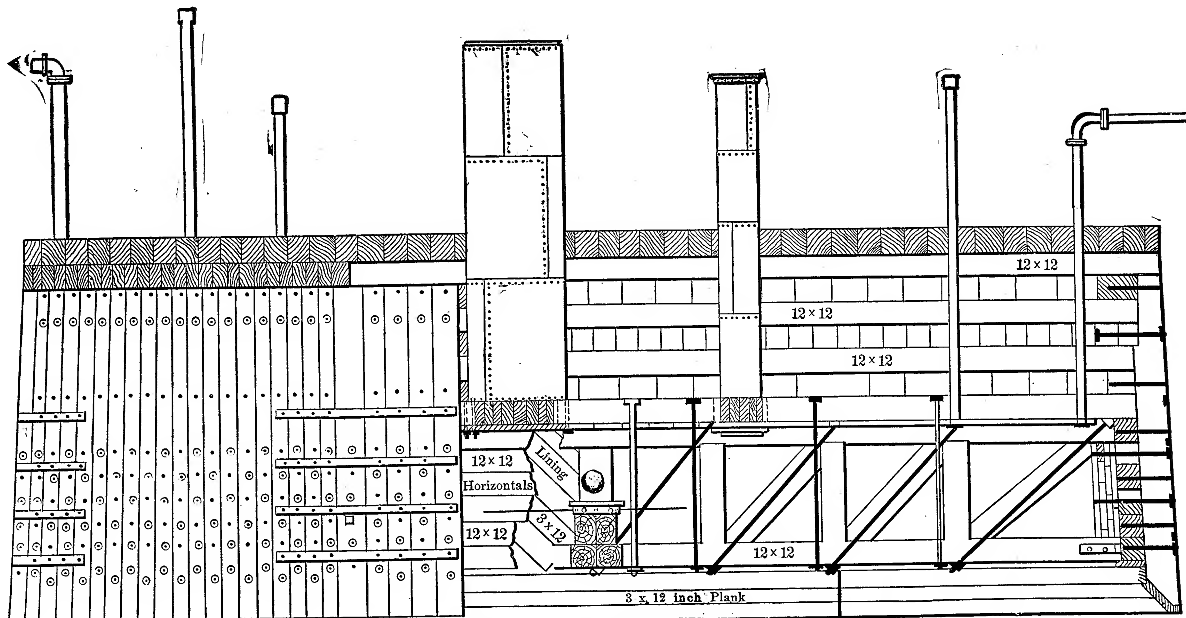
Extracts from the specification of Anderson & Barr, contractors, are as follows:

"We propose to sink the foundation 100 ft. below the bed of the shoal, if the material in the way of sinking the foundation is such that we can remove it by dredging. If the material is such that we must resort to the use of compressed air in order to remove it, we will sink the caisson 80 ft. below the low-water line, unless rock is encountered before that depth is reached. The foundation to consist of an iron caisson filled solidly with cement concrete. Concrete is to be made of 1 part of Portland cement, 2 parts of sand, and 4 parts of stone broken so as to pass through a 2-in. ring. The foundation caisson is to be built of cast-iron plates, with a bottom section of wrought steel. The total height is 155.5 ft. The wrought-steel bottom section is of cylindrical shape, 54 ft. diameter and 30 ft. high. On top of this is bolted a cast-iron conical section 20 ft. high, of 54 ft. lower and 45 ft. upper diameter. On top of this is placed the main body, which is of cylindrical form and of 45 ft. diameter, and which continues of even shape to the level of the base of the lighthouse tower. Through the whole body of the caisson parallel with its axis pass four water-tight steel cylinders of 9 ft. diameter, through which the ground is excavated from under the caisson.

"At 19 ft. high from the bottom edge of the caisson these cylinders widen out into irregular conical shapes, which end at 2 ft. 9 ins. above the bottom of the caisson in the circumferential cylinder shape and in a cross-bulkhead of 3 ft. height, which divides the area on the bottom of the caisson into four equal sectors of the circle.

Thus for 2 ft. 9 ins. height the bottom of the caisson consists of a single thickness of cylindrical outside plates, and bulkheads consisting of a single thickness of plate which divide the area of the circle into four equal parts, each of which is provided centrally over it with an open vertical tube 9 ft.

diameter, for the purpose of dredging its quarter compartment. The bottom circle of cylinder plates of 6 ft. height is of $\frac{3}{4}$-in. thickness. The cross-bulkhead plates are of $\frac{5}{8}$-in. thickness. These central straight plates, as well as the outside circle plates, are provided with stiffening brackets of $\frac{5}{8}$-in. thickness and 4-in. angle-irons. All the other plates of the wrought bottom section is of $\frac{1}{2}$-in. thickness, both the outside cylinder and the interior cones and tube ends. Four-inch angle-irons, running vertically the whole length of the section and 4 ft. apart, are riveted to the interior of the cylinder, and corresponding ones to the conical bottoms, and the dredging tubes are braced together by 4-in. angle-iron bracing in such form as to generally stiffen the structure against both outside and inside strains. Similar angle-irons brace those portions of cones and dredging tubes together which face one another. The top of the wrought section is provided with a 6-in. angle-iron on the inside of the cylinder, to which the cast-iron cone is bolted. This cone, as well as the cylindrical portion of 45 ft. diameter above the cone, consists of cast-iron plates of $1\frac{1}{2}$-in. thickness, and of such horizontal length as to make the circumference of 20 plates and of 5 ft. height. The plates are provided with planed flanges forming 6-in. depth of joint all around them, strongly bolted together with 1-in. bolts and nuts, and laid so that the vertical seams of different layers break joints. Lugs are cast on the plates, from which 4-in. angle-iron braces run to the nearest of the four central tubes. These latter are of a uniform diameter of 9 ft., made of $\frac{1}{4}$-in. plate in sections 5 ft. high, like the outside plates, and provided on top and bottom with 4-in. angle-iron outside rims for joining them by means of 1-in. bolts and nuts. The braces from the cast-iron circumference plates are attached to these rims; also the braces by which the tubes are braced to one another. For convenience of transportation and erection the cylinders are in halves, joined by 4-in. angle-irons and 1-in. bolts on the vertical seams. These tubes extend up to 2 ft. above the high-water line, and above that point the circumference plates are braced by turnbuckle bolts of $1\frac{1}{4}$-in. diameter.



ELEVATION OF OCTAGONAL CAISSON
Scale 1/4" to 2 feet.

Timber in Caisson
241,289 Ft.B.M.

FIG. 47.—PART ELEVATION, PART LONGITUDINAL SECTION, SHOWING INTERIOR DETAILS, SHAFTS, PIPES, IN POSITION.

[*To face page* 290.]

The whole interior of the caisson, including the tubes, will be filled with cement concrete, except that seven cylindrical vaults will be built in the floor of the towers. On the top surface of the concrete a cast-iron base of $42\frac{1}{2}$ ft. outside diameter and 2 ft. width, $1\frac{3}{4}$-in. thickness will be placed, on which the tower will be erected.

Total weight of the structure, 3,832,400 lbs.

Concrete about 10,000 cubic yards, and contract prices for the structure completed in place, $485,000.

The above is copied from the columns of the *Engineering News*.

For elevation, vertical section, and plan, see Figs. 40 and 40 (*a*).

29. The Cairo bridge across the Ohio River, near its mouth, was constructed in 1887–88 by the Union Bridge Co.

The superstructure of the bridge proper consists of 12 single-track steel spans, varying in length from 249 ft. to $518\frac{1}{2}$ ft. The piers supporting the longer spans rested on pneumatic caissons sunk 75 ft. below low-water. The masonry started 25 ft. below low-water and 10 ft. below the bed of the river. Length between end piers, $4644\frac{1}{2}$ ft. The approach on the Kentucky side consisted of 21 spans of 150 ft. = 3150 ft. and 4594 ft. of timber trestle. On the Illinois side the approach consisted of 17 spans 150 ft. and 1 span $106\frac{1}{4}$ ft. = 2550 ft. and 5327 ft. of timber trestle. All approach spans rested on piers composed of two steel cylinders filled with concrete and resting on piles.

The river bed is alluvial soil; some loose rock was found at a depth of 175 ft. It was determined to use the caissons for the foundations, as the apprehension of encountering logs, wrecks, etc., rendered the use of the open crib sunk by dredging risky and uncertain, and the loose nature of the material together with the rapid currents in floods precluded the use of piles. The dimensions of the caissons were 30 × 70 ft. and 26 × 60 ft. The height of the caissons and cribs were about 50 ft. The caisson proper was 16 ft., the pitch of the working chamber 8 ft., with two courses of solid timber forming the deck proper;

and on top of this six courses of timber of open-work crossing each other, and on this 34 ft. of open-work crib, similar to the upper 6 courses of what is called the caisson, leaving therefore 12-inch spaces between all of the timbers in every direction, except that the cross-braces divided the entire crib-work into a series of hollow prisms 7 ft. sq., extending from the top to the solid courses of the deck. The outside walls were covered with two courses of 3-in. oak plank, the inner layer placed diagonally and the outer layer vertically. The walls of the working chamber were V-shaped and built hollow. The whole was tied together by screw and drift bolts and spikes. The working chamber was lined with 3-in. plank, caulked and painted with two coats of white lead to prevent air leakage and aid in lighting the interior. The shoe of the caisson was made of iron plates $\frac{3}{8}$-in. thick and 36 ins. deep. The main shaft was only 3 ft. in diameter; supply shaft 2 ft. The air-lock was made of $\frac{1}{4}$-in. iron plates 9 ft. long, 6 ft. wide, and 7 ft. high, with circular ends 3 ft. radius, and was divided into compartments forming independent locks, and was placed 8 ft. above the deck of the caisson. The usual air, water and discharge pipes were used. The sand in the working chamber was removed by the Monson sand pump, somewhat different in design from the mud pump to be described presently, but similar in principle. The blowing-out process was also used, and to avoid a too great waste of air when the material was blown out dry, the pipe was extended below the cutting-edge so as to be under water. The maximum sinking in 24 hours was 10.63 ft., but the usual progress in clean sand was from 2 to 4 ft. daily; in some caissons it was only 1.1 to 2 ft. The greatest immersion was 94.2 ft. The calculated frictional resistance was from 597 to 715 lbs. per square foot of surface (the estimated resistance before sinking was 400 lbs. per square foot) at a depth in the sand of 86.42 ft. After several cases of paralysis and two deaths, a warm, comfortable room was fitted up, and also hot baths and coffee were provided, after which no further serious illness occurred. The temperature of the air in caisson was also cooled by passing through coils of pipe kept

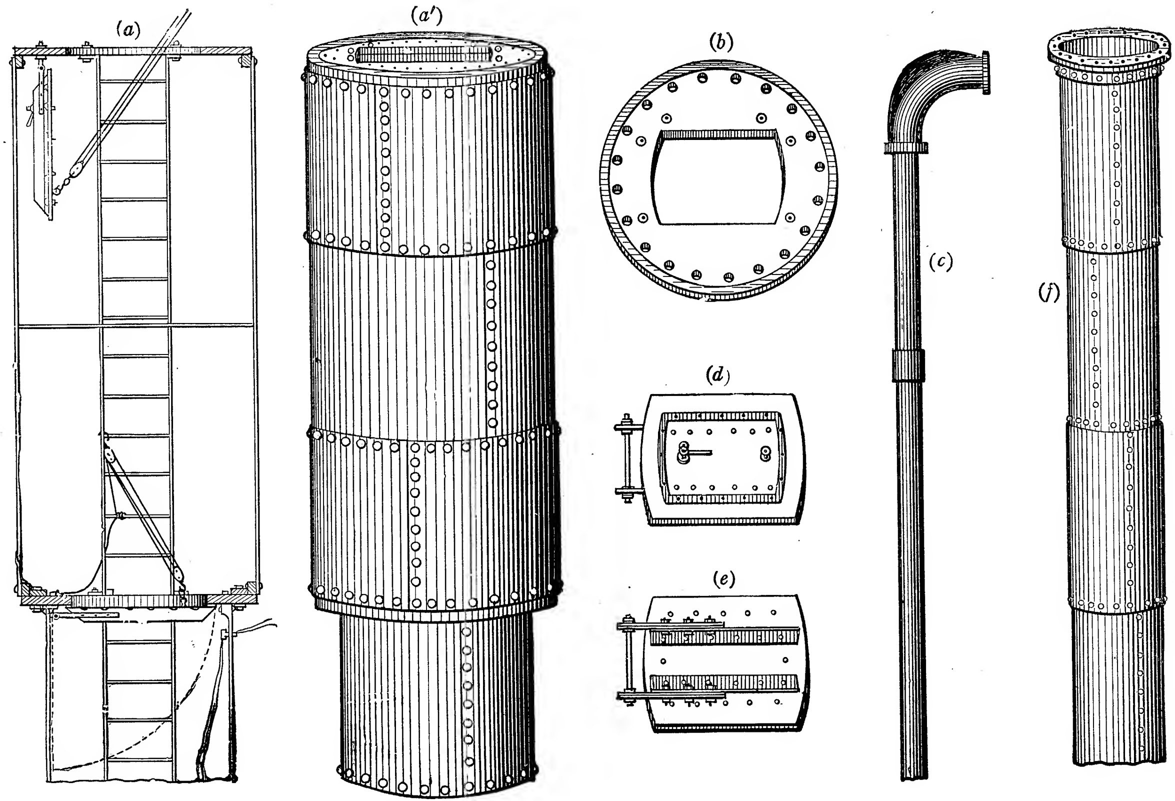


Fig. (*a*).—Vertical Section through Air-lock and Main Shaft.

Fig. (*a′*).—Outside View of Air-lock and Shaft.

Figs. (*b*), (*d*), (*e*). Details of Door.

Fig. (*c*). Discharge-pipe.

Fig. (*f*). Supply-shaft.

FIG. 48.—AIR-LOCK, SHAFT, PIPES, AND DETAILS.

[*To face page* 292.]

surrounded with cool water, lowering the temperature from 125° to 90°. The time of working the men varied from 8 hours to $1\frac{1}{2}$ hours per shift, allowing from 16 to 21 hours of rest during the 24 hours. Portland cement concrete was made, 1 cement, 2 sand, 3 broken stone. Louisville cement concrete, 1 cement, 2 sand, $3\frac{5}{8}$ broken stone. The piers for the approach spans consisted of two cylinders 8 ft. diameter, placed 18 ft. centres, braced together. Metal thickness $\frac{1}{2}$-in. plates, spliced on the inside. A pit was excavated 8 ft. deep, in the bottom of which twelve oak piles were driven; the pits were then partly filled with concrete, and the cylinders placed on the concrete. Concrete was then packed around the cylinders below the surface and also in the cylinders to the top, and was left about $\frac{1}{2}$ in. above. Over the top a steel plate $\frac{1}{2}$ in. thick was placed. After allowing 400 lbs. per square foot of surface on the caissons for frictional resistance, and after deducting the buoyant effect of the water and sand (respectively 22,756 cu. ft. and 78,000 cu. ft.) amounting to $9544\frac{1}{2}$ tons, from the total weight of 15,865.9 tons, the estimated weight on the foundation-bed was 6291.4 tons, or 3 tons = 6000 lbs. per square foot.

The precautions taken for the safety and comfort of the men certainly are to be highly commended. The writer hesitates to criticise the constructions of men of so much experience, knowledge, and skill; but he thinks that cutting up the space in the cribs with timbers in such numbers separated by only 12 ins. of space is a faulty construction, and must necessarily require either a great deal of labor and care to fill around and under so many square timbers,—round logs for cross-braces would to some extent remedy the objection,—or if the work is carelessly done there must exist many hollows and open spaces. The position of the air-lock near the bottom can hardly be recommended.*

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* Only the caisson proper is shown in Fig. 51. The roof consists of two solid courses of timbers, and six courses of timbers built open, as shown in Fig. 51, (*c*). The crib can be built on top of the caisson to any desired height, 34 ft. in this case, and is built open, as shown in Fig. 51, (*c*). The V-shaped walls can be built hollow and filled with concrete, or they can be solid built with timber, as shown on the right in Fig. 51, (*c*).

These parties, however, have put in more caissons than almost any others. The success which has attended their works certainly cannot be criticised, and it must be presumed that they consider it economical and satisfactory in every respect. See Figs. 50 and 51, (*a*), (*b*), and (*c*).

29½. Having described briefly the caissons of the above large bridges, we will now consider in somewhat greater detail the design and construction of the caissons used by the writer on several large bridges, as the Susquehanna Bridge, Havre de Grace, Md., the Schuylkill Bridge, Philadelphia, Pa., and the Tombigbee River Bridge, Ala. The caissons were nearly as large, and the depths sunk were about as great; hence the details of construction, methods of sinking, etc., would be equally applicable to any of those already described with few modifications of minor importance, while in connection with these the descriptions will be based upon actual experience, as the caissons were designed by, and the work executed under the direct supervision, of the writer; but as applied to the others they would be purely a compilation from the descriptions of others.

The design and construction of the caissons were the same for the three bridges. There were 5 caissons in the Susquehanna River Bridge, varying in dimensions from 63.27 × 25.93 ft. to 78.19 × 42.27 ft. and a general thickness of roof of 8 ft. The widest caisson had a roof 10 ft. thick; these were built solid, of courses of 12 × 12 in. pine timber. At the Schuylkill there were two rectangular caissons 65.5 × 23.5 ft., one octagonal caisson 50 ft. in diameter of circumscribing circle for pivot pier, and one nearly square caisson 44 × 45 ft. for a U-abutment; the roof of this latter was 10 ft. thick, of the others 8 ft. thick—the depths sunk varying from 40 to 90 ft. below low-water.

At the Tombigbee Bridge there were two rectangular caissons 45 × 23 ft. and one octagonal caisson 24 ft. diameter for the draw pier. These caissons were sunk only about 33 ft. below low-water, but the excavation was continued about 9 ft. below the cutting-edge of the caisson to a point about 42 ft. below low-water and 82 below high-water.

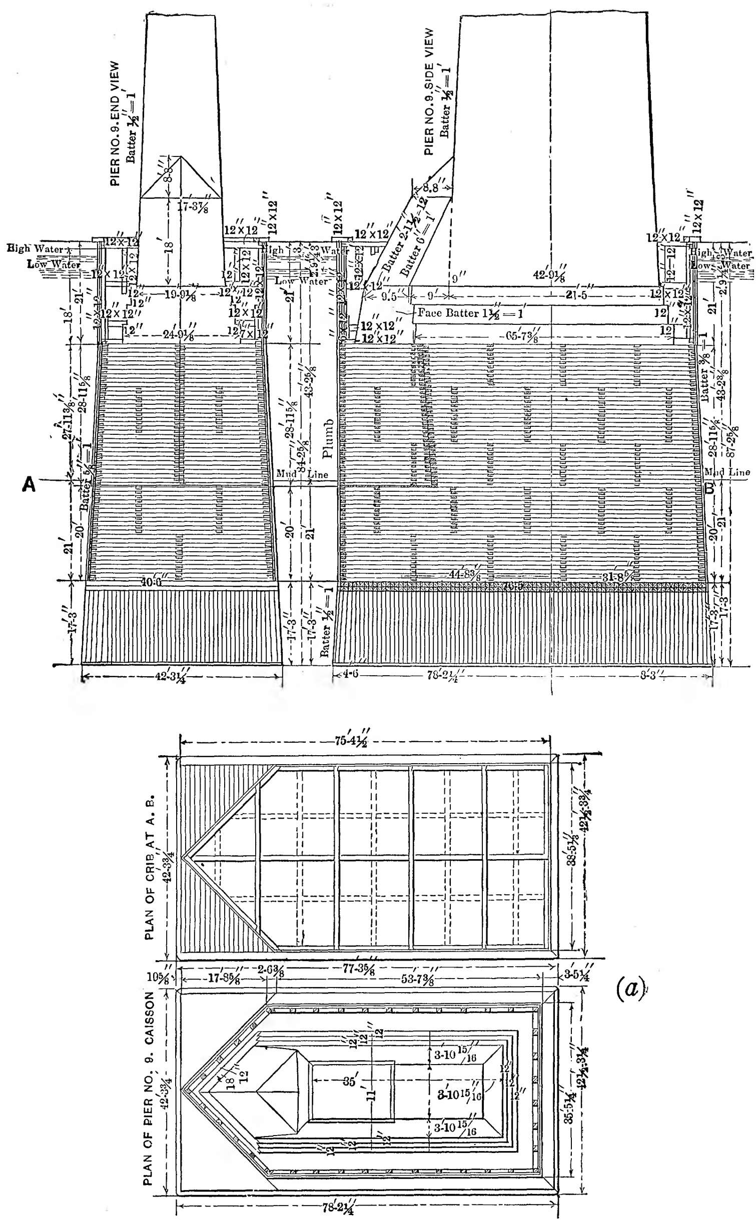


FIG. 49.—DESIGN OF PNEUMATIC CAISSON AND CRIB WITH POINTED END; ALSO, LOWER PART OF MASONRY PIER, WITH CUTWATER.

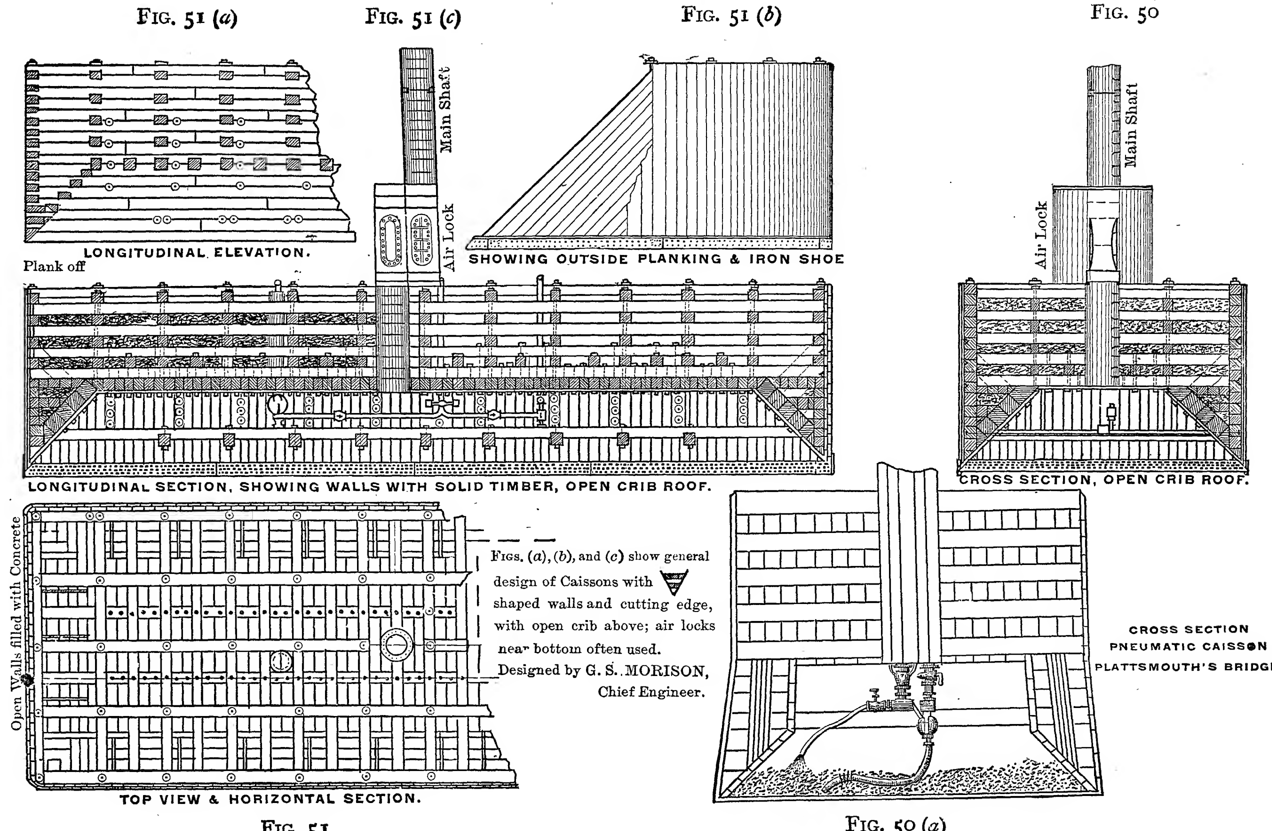
[*To face page* 294.]

30. On all of these caissons cribs were constructed and filled with concrete, varying in height from 20 to 40 ft. of the same horizontal sections as the caissons at the top, which was about 20 ins. less in each dimension than that given above, as the caissons were 15 ft. high, and had a batter all around of $\frac{5}{8}$ in. to each vertical foot.

31. Coffer-dams were constructed on top of the cribs from 20 to 40 ft. high, according to the depth of the water in which the masonry of the piers was constructed.

32. A description of one caisson, crib, and coffer-dam will answer for all, with a few modifications for the octagonal forms required by its shape. The descriptions will be better and more easily understood by reference to Figs. 43, 44, 45, 46, 47, 52, and 53. The plates show horizontal and vertical sections, plans, details, etc., of caissons, cribs, and coffer-dams. The caisson was constructed by first building a solid wall of five or six courses of timber, $12\frac{1}{2} \times 12$ ins. cross-section, surrounding the required space, and built with the proper batter. On the outside of this, timbers, 12×14 ins. $\times 14$ ft., were placed all around, extending 2 ft. below the timber-wall and 6 ft. above; the lower edges of these pieces were cut to a bevel, the lower cutting-edge being 3 inches thick. On the inside of the wall three courses of 3-in. plank were placed, crossing each other diagonally, and on the inside of this a single course placed vertically—for convenience of calking. The courses of plank were cut to a level with the top of the wall and reached to within one foot of its lower edge. The whole was then bolted together by both screw and drift-bolts, as shown in the drawings. Each layer of plank was also spiked with two spikes, $5\frac{1}{2}$ inches long, to each lineal foot of plank. Then plank was also spiked in one layer on all interior surfaces below the courses of plank above mentioned. This completed the walls of the working chamber. The deck courses were then placed between the verticals projecting upward and resting on top of the timber-wall and the four courses of plank on the inside, which gave a 2-ft. bearing on all sides. The arrangement of the courses was as follows:

1st. A course of timbers in one length, laid transversely; 2d, a course diagonally, of varying length; 3d, another course laid transversely, of single length. At the top of this course a 2-in. shoulder was formed in tne verticals; 4th, a course laid longitudinally, resting on the shoulder; 5th, another transverse course, in single lengths; 6th, a diagonal course in varying lengths; the verticals were cut off on a level with the top of this course. The 7th and 8th courses were transverse and of single length, reaching from out to out over the tops of the verticals. These latter were bolted by screw and drift-bolts to the deck-courses, as shown on the drawings. The deck-courses were all bedded in a good bed of cement-mortar and a thin grout poured into the intervals between the timbers of the same course; this interval being about $\frac{1}{4}$ inch. Each stick was bolted to the course below by drift-bolts, 1 in. square or round, at intervals of about 5 feet. The underside of the roof was lined with 3-in. plank. The whole interior was then thoroughly calked with oakum. This extended the full thickness of the plank, and when properly done the oakum compressed would be harder than the timber itself. The ends of all bolts and spikes were also covered or wrapped with oakum, the heads and nuts bearing hard against the oakum. The shafts, pipes, etc., were built into the roof, all spaces around them filled with mortar. In all caissons a longitudinal truss was constructed, resting on and fastened by iron straps and bolts to the end walls. This truss was about 6 feet deep, the upper and lower chords composed of two pieces 12 × 12 in. timbers. The web-members, both vertical and diagonals, were composed of timber struts, and diagonal rods $1\frac{1}{2}$ in. in diameter, these latter extending through the first deck-course. This truss formed a strong stiffening rib for the roof, and also braces for the end-walls, and in addition affording a broad bearing surface for blocking or for the earthy material. Cross-braces were placed between the bottom chord of the truss and the side-walls. These were of timber, either 12 × 12 ins. or 12 × 16 ins., depending upon the length required. In addition, at each strut-brace, iron rods, 2 ins. diameter, with



GENERAL DESIGNS OF PNEUMATIC CAISSONS.

Figs. 51, (a), (b), and (c) used in foundations of Bismarck and Cairo bridges. Open Crib-work built to any desired height.

[To face page 296.]

swivels, extended across the caisson and through the side-walls. Details of these rods are given in Fig. 52 and 53 (*a*).

33. This completed the caisson proper. The caissons were built partly on shore, supported 5 or 6 feet above the ground on blocks of timber. Generally only one or, at most, two courses of deck-timbers were placed, until the caisson was launched. After the interior was completed and calked, launching-ways were built under the caisson, and the caisson supported on a number of screw-jacks; the cradles or sliding-ways were adjusted, the jacks lowered, so as to let the caisson rest on the cradles, and when everything was ready the caisson was launched, then floated to its proper position, where it was completed. It is not necessary to put a bottom to the caisson when deep water is accessible; it causes ultimate delay and trouble to remove it. The caissons finished as above described, with only 2 deck-courses, would be immersed only about 8 or 9 feet. With a good bottom, they would float in about 3 or 4 feet of water. Figs. 47, 52, and Figs. 44, 45, 46 show details, section, and plan of an octagonal caisson, the interior struts radiated from a centre-post and rested against the sides; the iron-rods radiated from a centre-collar of iron, and passed through the angles. In all the caissons, iron-bars, $2\frac{1}{2} \times 1$ in. $\times$ 8 or 10 feet, were bent around the angles on the outside and bolted to the timbers, 3 or 4 straps being placed at each corner.

34. The advantages of these forms of caissons are evident. The walls of the working chamber are strongly and firmly connected with the roof of the caisson, forming stout cantilevers, thereby relieving the pressure on the braces, as was evidenced by the fact that in no case were the wedges at the ends of the strut-braces in working chambers crushed, even under very trying circumstances, as when the air escaped suddenly from the caisson; and all parts acting together local and excessive strains never caused any springing or leaks, nor was there any creaking or cracking of timbers to alarm the men. The walls of the working chamber are so constructed that the men had easy access to the cutting edge, and at the same time broad hori-

zontal surfaces are provided so as to obtain many square feet of bearing surfaces at any part of the caisson or entirely around, in addition to the bearing surfaces under the centre truss. These conditions are of great advantage in many cases. The caisson can be better kept level, or in case of careening can the more easily be brought to a level, and there is less danger of settling until everything is ready, as the material can be left under these bearing surfaces; and in short the caisson can be controlled and regulated much better than when the walls ot the working chamber are V-shaped. The latter design of caisson is shown in Figs. 50 and 51, (*a*), (*b*), and (*c*).

35. The only accident that happened during the construction of the Susquehanna bridge was caused by the neglect of one of the foremen, and as much can be learned from accidents this one will be briefly described. The largest caisson had reached a point within seven feet of the rock at its highest point, but was twenty-eight feet above the rock at its lowest point. Owing to the softness of the material through which we were sinking, it was necessary to stop concreting in the crib to avoid too much weight; the coffer-dam had been constructed on the crib, but had not been braced on the interior. At this time the top of the crib was only a few inches above the surface of the water; the pockets of the crib were empty for a considerable depth. Without observing these conditions the foreman, being ready to sink the caisson, lowered the pressure at a time when the tide was at its highest. As the caisson settled the water raised a few feet above the crib; the pressure caused one side of the coffer dam to be forced inwards, the water rapidly filling the crib and adding about 14,000,000 lbs. of weight; the caisson sank suddenly until one end rested on the rock, then careening, settled at the other end until a sufficient bearing on the roof of the caisson stopped it. Seven men were in the chamber at the time; they were fortunately either in the shaft or near to it, and ascended to a place of safety; fortunately the lower door was closed at the time, and they could not enter the lock. The upper end of the shaft sank under the water, allowing the air-lock to be filled with water; pipes were broken

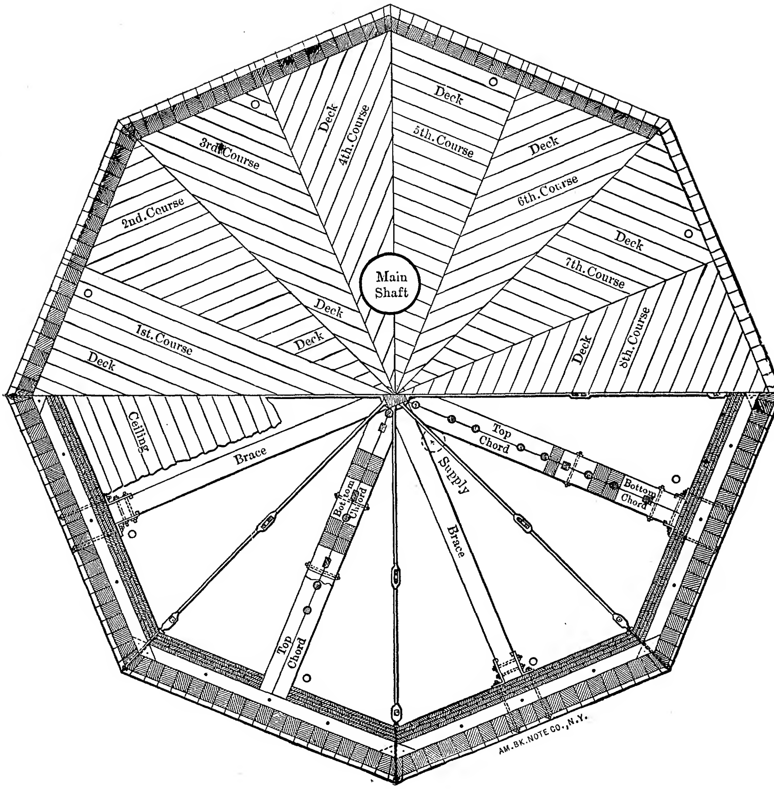


FIG. 52.—HORIZONTAL SECTIONS OCTAGONAL CAISSON FOR PIVOT PIER, SCHUYLKILL RIVER BRIDGE, B. & O. RY., SHOWING DETAILS OF CONSTRUCTION.

[*To face page* 298.]

off, and leaks were caused in the main shafts. This air following the shaft made the water boil up furiously around the top of the lock. Four large air compressors were started at once, all pipes were plugged up above the surface; and notwithstanding the large quantity of air that was being forced into the caisson, the leaks in the shafts were so great that the water was gradually rising. The men tore their clothes and stuffed them in the openings. Great difficulty was encountered in getting a dam around the mouth of the shaft; but by the use of planks, tarpaulins, etc., the bubbling and boiling was stopped, and a dam of cement in bags was made, the interstices packed with oakum dipped in mortar. We at last were able to bail the water out of the lock, and the men were released, after having been confined in their perilous position for about eight hours. When the necessary repairs were made and the air again forced into the caisson, it was found that no leaks existed in the caisson proper; that the only damage, outside of broken valves, pipes, etc., had occurred where the caisson had brought up hard on the rock; the lower ends of the verticals had been crushed off to a height of about two feet around one corner of the caisson. This was of no moment, and the work proceeded at once The rock was blasted to a depth sufficient to level the caisson, which was accomplished without any special trouble. We can learn from this, 1st. That the caisson should not be sunk until a careful examination is made to see that everything is ready; 2d. Always keep the top of the main shaft well above the surface of the water; 3d. Always bring the men out of the caisson before sinking the caisson, and 4th. The necessity of having ample steam-producing capacity and also reserve compressors, to supply large quantities of air, connected up and ready for work at a moment's notice. Anything short of this amounts to gross negligence or carelessness. Air compressors are often stopped for a greater or less time (and men left in the chamber) for small repairs, want of steam, or other causes, or in case of any alterations in the air connections of pipes or shafts. The men should invariably be taken out of the caisson; many lives have been lost in recent years by a failure to do so.

36. In some large bridges the masonry is commenced on top of the caisson, without using cribs or even coffer-dams; there is always danger of delay and extra cost. The crib can be dispensed with, but a coffer-dam should always be constructed, as it is vastly cheaper to construct a dam while the work progresses than to build one after the deck of the caisson has disappeared under water.

37. A crib is only necessary for rapidity of sinking, and is a matter of economy; a timber or iron-cased crib will not cost more than one half to one third that of masonry per cubic yard; the crib can be built up rapidly and with relatively small expense in calking will be sufficently water-tight; occasional bailing or pumping will keep the leaks down. A crib of this kind is really a solid single-wall coffer-dam, well braced on the interior.

Sometimes cribs are built open, leaving 12-in. spaces between the courses of timbers; the transverse braces passing between the courses having likewise 12-in. spaces between them. There is no economy in this, as the spaces are to be filled with concrete (see Figs. 50 and 51). As water circulates freely in the crib, much of the concrete will either have to be placed under water or subjected to the action of the water before it has had time to set, preventing sound, solid work and causing waste of good material. The work cannot be altogether satisfactory, and as the concrete should be packed under and between the cross-timbers, hollow spaces will necessarily exist. This can be avoided largely by using round logs, stripped of bark, for the cross-braces; they are equally as good, and would cost somewhat less than braces sawed square. A crib thus constructed practically divides the mass of concrete into isolated prisms or columns of concrete.

Solid walls, either calked or not, are used with solid or open-work cross-walls, or braces; the same objections occur in this case, so far as isolating the columns of concrete.

In the cribs used in the structures now being described, both of these objections were to a great extent removed. The outside walls were built solid and calked; the cross-walls were built solid for a few courses of a height one third or one fourth that of the crib. The positions of the cross-walls were then shifted to the middle of the pockets below, and built up solid for a like height, then shifted vertically over the lower walls, and so on alternating 2 and 3, and 3 and 4 in number. In this case it was only necessary to pack the concrete under a few timbers, which could be cut on a bevel or be round; the various columns of concrete were consolidated into practically a homogeneous and united mass. The water could be kept from any layer as long as desired. The side-walls were dovetailed at the corners, and the cross-walls were dovetailed into the timbers of the outside walls. All timbers were drift-bolted together with 1-in. round or square bolts 22 ins. long, spaced about 5 ft. intervals—the courses of timber breaking joints. These cribs were planked on the outside, the plank placed vertically in lengths of 5 to 7 ft. and spiked; this kept the calking in place—otherwise of no special advantage. At the 5th or 6th course of timber from the top, iron bolts 2-ins. diameter, with a large eye on one end, were placed through the outer walls of the crib; these were for connecting the vertical rods of the coffer-dam. Drawings, Fig. 43, fully illustrate the construction of the cribs; these were square-ended. Pointed-end cribs can easily be constructed when desired, and should be pointed, if they extend near to the surface of the water, so as not to obstruct the current too much (see Fig. 49). The tops of these cribs were from 15 to 30 or more feet below low-water, and did not justify the additional amount of material and costs as it would have also necessitated longer caissons and considerable extra expense in the sinking.

38. The design of coffer-dams used was simple, strong, and efficient. When the height required was over 20 ft., they were built in two sections similar in every respect to each other. A 12 × 12 in. sill was placed on the walls of the crib, overlapping it by 3 ins. on the inside. Vertical pieces 12 × 12 ins. were erected on these at intervals of 4 or 5 ft., connected with the sill by mortise and tenon, and then caps mortised and tenoned to them; cross-pieces placed across the top projected outward over the iron eye-bolts in the crib and long iron rods 2 ins. diameter with hooks at one end and threads at the other were hooked to the eye-bolts and passed through holes in the cross-pieces, on the upper ends thimbles or sleeves with right and left hand threads were screwed, pressing the coffer-dam hard to the crib. The sleeves were used instead of nuts, so as to connect other rods for the upper sections of the dams; the usual nuts were used on the top of these bolts. A double course of 3-in. plank was then spiked to the uprights—the first or inner course laid diagonally, and the outer horizontal for convenience of calking: the entire outside was well calked. Two tiers of braces on the inside were sufficient for a section 20 feet high; strong cleats were spiked to the cross-walls of the crib to brace the bottom sills. Plans, sections, and details are shown in Fig. 43, (*a*) and (*b*). The upper section was constructed in the same manner. This construction answers well for heights from 40 to 45 ft. The octagonal cribs and coffer-dams are different only in the struts and rods for bracing which radiate from centre-posts. The corner-posts of all dams were made in two pieces, and bolted together; when these bolts were removed and the iron rods unhooked, the sides and ends were free to separate. It was intended to use these on other cribs, but it did not prove either economical or practicable. The only coffer-dam that gave away was caused by the accident already explained.

Fig. 51, (*a*), (*c*), and Fig. 50 show another form of caisson, crib, etc., often used, open-wall cribs being employed.

Article L.

CAISSON SINKING.

39. The construction of the caissons, air-locks, size, and kind of pipes, machinery, connections, etc., having been described, it only remains to explain briefly the methods used in excavation, sinking caissons, and filling the working chamber with concrete. It may be stated that in general all materials that are too large to pass out through the pipes have to be

Fig. (b) —Cross section.

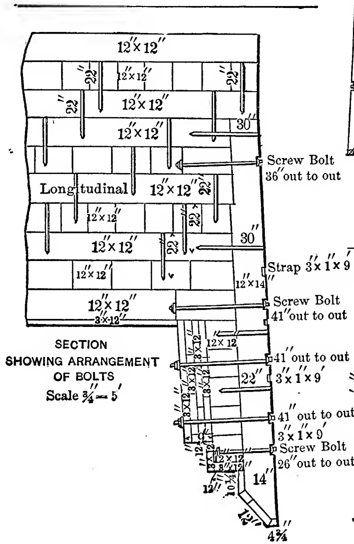
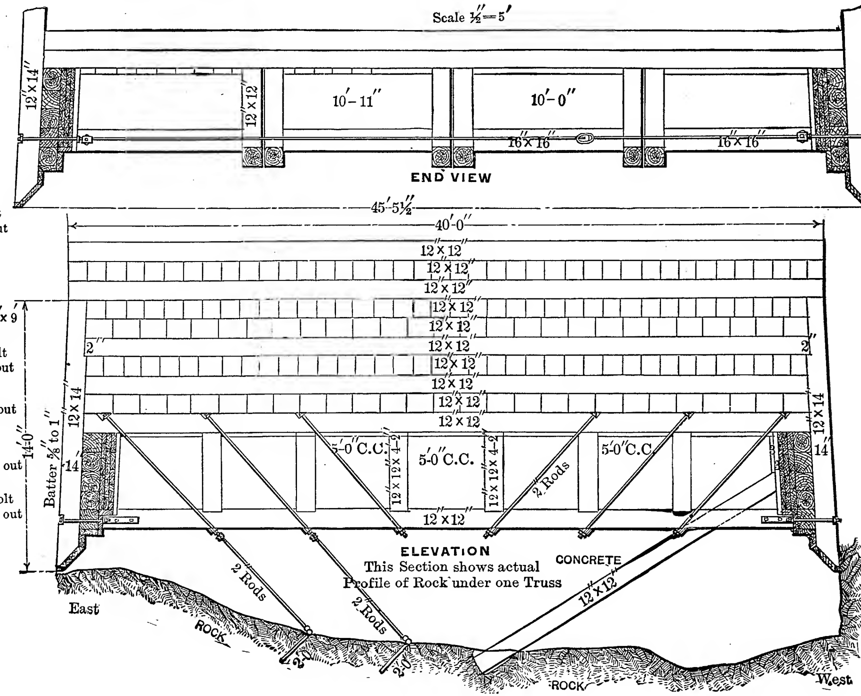


Fig. (a).



Longitudinal Section.

FIG. 53.—SQUARE CAISSON FOR MASONRY ABUTMENT.

[To face page 303.]

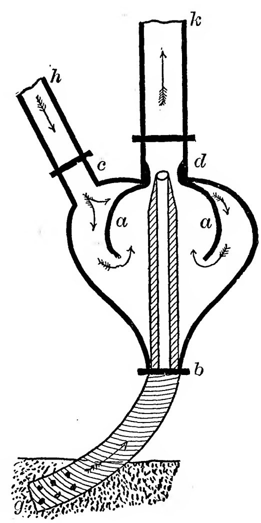
carried out through the main shaft in buckets or bags. There are patent buckets, which slide through a shaft left in the caisson, being raised or lowered by machinery. When the bucket is lowered into the working chambers by the proper adjustment of valves and pipes, doors can be opened into the working chamber, large bowlders, sticks of wood, and other débris can be thrown into the bucket, the doors closed, air pressures equalized, and the bucket with its load lifted out. The arrangement is simple and efficient, but has never been generally adopted. In the writer's experience the larger bowlders and pieces of crushed rock were generally piled on platforms resting on the truss and braces and carried down with the caisson, mainly removing from the interior the sand, gravel, etc. The bowlders were ultimately used in the concrete or rubble-work in the chamber. There are some objections to this, as the men are inconvenienced in moving about, and have to work under heavily loaded platforms, which involves some danger. It causes some delay and expense, but on the whole is probably more economical than breaking up the bowlders, removing them from the caissons, and again putting them back in the form of concrete.

40. The removal of the sand, gravel, and mud can be effected by the sand pump, mud pump, or by the blowing-out process, each of which will be briefly described. As has been mentioned, a number of discharge pipes were built into the caisson, extending through the deck. Sections of pipe 8 or 9 ft. long are screwed on to these at the bottom, reaching down into the material, the lower end bent at right angles. A small wooden paddle is pressed against the end by the air when the valve is open; the material excavated is shovelled in a pile around the lower end of the pipe; when the paddle is removed the air forces the material up and through the pipe with great force. At the top an elbow or goose-neck of chilled iron two or three inches thick is fastened. This discharges the material outward and downward. These elbows are rapidly cut through by the sand and gravel, requiring frequent renewals.

For details of shafts, air-locks, pipes, etc., see Fig. 48.

The process is simple, but requires great care in feeding the material to the pipe to prevent its choking up. A dense fog always sets up when the pressure is lowered, and often water rises in the caisson, and much air is used, taxing the air compressors greatly. Notwithstanding these objections it is largely used.

41. The mud pump and sand pump do not differ materially in design, nor at all in principle. The mud pump will be alone described. It consists of a pear-shaped cast-iron vessel about 15 ins. in diameter and length, which has a hemispherical lining, *a a*, connected with the top; also three openings into it, *b*, *c*, and *d*, to which hose or pipes can be connected. *bg* is called the suction pipe, *ch* the supply pipe, and *dk* the discharge pipe. *bg* is a long hose, so as to be moved freely about; its lower end has an iron strainer to prevent any large material or sticks, etc., entering. Its upper end is screwed into the bell and has a hollow, conical-shaped point, which reaches into the neck of the discharge pipe, which also tapers slightly, so that the annular space between the two can be either widened or narrowed. Water is forced by a large pump down through the supply pipe, and impinging on the iron lining is scattered around it, and then passes upward through the annular space, and upward in the discharge pipe, *dk*. This creates a partial vacuum at the end of the supply pipe. The sand, mud, and water are thus drawn up into the discharge pipe, and are discharged at the top. A large quantity of material can thus be removed without decreasing the air-pressure, but the material is required to be cut up fine and mixed with water.*



MUD OR SAND PUMP FOR REMOVING MATERIAL FROM WORKING CHAMBERS OF PNEUMATIC CAISSONS.

* The Monson sand-pump has a vertical section almost oval, is made of cast iron with wrought-iron bands, horizontal section is nearly circular. The supply-pipe enters the pump near the bottom; there is no inner lining, otherwise the design and construction is similar to the above-described mud-pump.

42. In making the excavation, the material should not be removed from under the shoulders until the middle space has been excavated to the depth of 2 or more feet below the cutting-edge, so as not to leave the caisson unsupported for any great length of time, and not at all under the lower side, if the caisson is out of level. When everything is ready, the men should be brought out and the caisson lowered by gradually reducing the pressure. When the resistance to lowering is very great, requiring a great reduction of pressure, one man generally remains in to see if any serious leaks occur or any great inflow of material takes place, so as to signal for the pressure to be put on. He could readily ascend the shaft if necessary.

43. As has been stated, the borings indicated a very great difference of level in the rock-bed, being from 15 to 20 ft. in the length of the caisson in some instances, and that blasting from the surface had proved impracticable at any reasonable cost. During the sinking, as the caisson approached the highest part of the rock, constant soundings were made with an iron rod, to avoid the danger of coming suddenly on the rock at any point, and when the highest point of rock was reached our principal difficulties commenced. There was but two courses open: either to blast the rock and sink the caisson to the level of the lowest point of the rock, or to hold the caisson where it was and carry on the excavation below the cutting-edge, then build up with concrete under the caisson, and then fill the working chamber. The latter plan was adopted, as the rock was very hard and only small charges could be used, which would have required a long time and added enormously to the cost of the work. The great danger in excavating below the cutting-edge arose from the fear of the caisson careening and settling out of level. This danger was obviated, however, by cleaning out sections of about 10 ft. square, one at a time, leaving the rest of the caisson well supported. The rock at the bottom of these pits, if sloping, was blasted to an irregular surface, forming depressions and elevations. The concreting was then commenced and carried up to the cutting-edge and packed under the shoulders. Another section was

then completed in a similar manner. Where the depth to the rock did not exceed 5 or 6 ft. no trouble arose; but in greater depths the material under and outside the cutting-edge would cave in, endangering the safety of the caisson by a sudden escape of the compressed air, called "blow-outs." This would frequently happen in sand and gravel, but seldom in clay or silt. To avoid this difficulty the pits were lined with frames and sheeting, as in sinking shafts into the ground. These timbers had to be cut of the proper lengths and carried, one by one, down the shaft; but by this means pits 12 to 15 ft. deep were sunk and filled with concrete. In sand and gravel it was often impossible to hold the material, and the framing or bulkheads would break in, followed by much inflow of the material and escape of the air, but, gaining little by little, the entire side would sooner or later be sealed up. This difficulty in sand and gravel arises from the fact that the pressure cannot be kept up greater than that due to the depth of the cutting-edge below the surface of the water, as the escape of the air is so great. In clay or silt the material itself is air-tight or nearly so at that depth, and the pressure can be raised to almost any extent. Caving in also occurred in this material to some extent when unsupported, but it could be easily held in place. In the case of the caisson to which the accident happened, as already described, we were compelled to blast the rock to a depth of about 7 ft. around a part of the caisson in order to level it. This still left about 13 ft. to be excavated below the cutting-edge at the other parts, which was done as above described. Having in this manner constructed a wall of concrete entirely around the caisson, the material enclosed was then removed. Blasts were put in all the sloping surfaces, bringing the entire surfaces to a series of depressions and rises in both directions, in order to prevent any danger of sliding. No attempt, however, was made to cut the rock to a level or even to a series of steps, the surface being simply very much roughened. Some engineers have drilled large holes in the rock and inserted iron rods projecting a foot or more above the rock in order to prevent sliding.

44. The filling of the air-chamber with concrete was then proceeded with. All the concrete was mixed in batches, using about a barrel of cement to the batch. This was mixed by hand on a platform above and was passed through the supply-shaft, which was simply a long air-lock formed by a door at top and bottom. When the signal was given the concrete was mixed and immediately shovelled into the shaft, the lower door being closed and further supported by a timber strut. When a sufficient quantity had been thrown in—from $\frac{1}{2}$ to 1 cubic yard—the upper door was closed, the air equalized, and the lower door opened, the concrete dropping on a platform. It was then carried in barrows, deposited in place, and rammed. Before throwing the concrete into the shaft several buckets of water were thrown in, and also after throwing the concrete in. The water prevented the cement from adhering to the shaft and from heating and setting too rapidly when the compressed air entered the shaft; otherwise the shaft would be blocked, and it would be difficult to clear it again. The hot air of the chamber, unless a plenty of water is used, causes the cement to set before it can be properly handled. It requires great care and a concrete rather dry and mixed with very small chips of stone to pack close against the deck of the caisson. It is better to leave one or two sections of shaft in place. The upper sections can be removed and used over again.

45. There is nothing of special note in the Schuylkill River caissons except their great length as compared with the width, which was required by the line crossing the stream very obliquely. The abutment caisson was nearly square. No crib was used, but a high coffer-dam was constructed on the caisson: this was filled solid with rubble masonry, one man stone bedded in concrete; the air chamber was filled with concrete. As this caisson had to support the thrust of a heavy mass of earth resting on the swamp, timber strut braces and large iron tie-rods were used in the working chamber to prevent sliding, and the bed was given a slight slope against the direction of the pressure. (Fig. 53, (*a*), (*b*), and (*c*).)

46. The points especially worthy of notice in the Tombigbee River bridge was the nature of the material on which the structure was built, and an accident that happened to one of caissons, from which some useful information can be obtained.

The site of the bridge was inaccessible; the river itself being the only avenue for transportation, and this alternating between extreme high and low water. All materials except timber had to be transported long distances. Brick and shells were used in the concrete. The material underlying the water was a shifting sand, resting on a silt intermixed with irregular bowlders, or broken layers of a bluish-white marl. There was 23 ft. of water at the lowest stage; but sudden rises of 35 to 40 ft. often occurred, and at irregular and uncertain periods. A simple crib or open caisson, resting on the bed of the river, would inevitably have scoured out; nor could piles be relied upon, as owing to the irregular layer of marl, through which they could not be driven, some would have scoured out. A coffer-dam would have been required of great height, and liable at any time to be scoured out or flooded; and in addition, the varying depths of the borings left it uncertain as to the proper depth at which the structure should be founded. For these reasons the writer determined to sink pneumatic caissons, as then all doubts and difficulties could be settled at the proper time. The octagonal caisson was sunk through 23 feet of water, 9 feet of sand, silt, and patches of marl, and the excavation carried about 9 feet below the cutting-edge in silt. It was found impracticable to sink the caissons farther, although the entire air-pressure was let out of the caisson. This indicated an unusual frictional resistance on the outside; doubtless due to the marl bowlders bearing strongly against the sides of the caisson. The pier, however, at this time was only about one half completed; but with this large and well-defined frictional resistance, and the fact that borings indicated an almost unfathomable depth of silt below, it was determined to build at that point. The caisson was filled with concrete resting on the silt; this material was so soft that a rod four or six feet long could be readily pressed into the material. The writer's experience with driving piles, and their great bearing capacity in that kind of material, together with the fact that on the same river screw-pile piers constructed by him, having only 100 sq. ft. bearing to the pier and carrying spans 150 ft. long, carrying the heavy loads of the present day, had stood for nearly twenty years, gave confidence, as this pier would have fully 1000 sq. feet of bearing, although it would carry heavier piers and longer spans. The weight at the time the caisson was stopped was 1,684,500 lbs.; and as the caisson did not rest on anything at the bottom, the entire cutting-edge being cleared in order to sink the caisson, if possible, and the air-pressure entirely relieved, it was a clear case of balance by friction. And as the exposed surface was 1200 sq. ft. the frictional resistance must have been 1400 lbs. per square foot. Then concrete was packed under the cutting-edge and shoulders on the lower side, and the pressure again lowered; the weight now acting with a lever-arm of about 10 feet. The frictional resistance on about one half of the exposed surface would be acting with an arm of about 20 feet or more. But the caisson did not settle a particle; this seemed to be conclusive as to the ultimate bearing of the foundation. The completed structure, including the rolling load on the bridge, weighed 4,374,500 lbs.; area of base of caisson, 1148 sq. feet; bearing resistance of foundation-bed, not considering any allowance for friction = 3810 per sq. ft.; and allowing 1,684,500 lbs. for frictional resistance, the pressure on the silt is 2343 lbs. per sq. ft. This is not an unusual pressure for this material, as seen in paragraph 306, Part I, Table No. 6.

This is particularly mentioned as a safe load at that depth, on the softest material that can be called solid or earth. This bridge has been in use now for over six years. The piers were built of brick, and carry 275-ft. spans. Such spans on brick piers are somewhat unusual. The brick was hard, sound, well burnt, and laid in cement mortar.

47. One of the rectangular caissons 45 × 23 ft. × 14 ft., with a crib 20 ft. high, partly filled with concrete weighing 800 tons, simply resting on the bottom, was lifted by the water in a rapid rise of the river; and although well secured to a num-

ber of clusters of piles driven around it, and swung askew of its proper position and dropped 10 ft. down stream, it could not be pulled back into position against the current, and had to be flooded where it was. Sinking somewhat suddenly, the material at the upper end was scoured out; this swinging instantaneously into the eddy under and at the down-stream end collected into a mound, and when the flood subsided the caisson was found in an inclined position at an angle of about 35° or 40°. A contract was made with parties accustomed to lighter vessels across the bar below Mobile to lift the caisson into position. The first difficulty was in getting chains under it. This, however, was ultimately accomplished, the lighters lowered, necessary connections made, the water pumped out; but the caisson did not lift, the largest iron chains snapping and breaking. Failing in this the concrete was blasted out of the crib; the caisson did not float until air connections were made and air-pressure put on, when it rose suddenly. It was then located and the work proceeded to a finish as usual, but many thousands of dollars had then been wasted.

The first lesson to be learned from this accident is that it is unwise to attempt to resist the action of such rapid and high rises in rivers. Had this crib been flooded in the earlier stages of the rise, and had we waited patiently for the fall of the river, both time and money would have been saved; and, second, it is a waste of time and money to endeavor to lift such structures in place. It is far better to lighten the load, and let natural laws and forces aid in the floating of the structure.*

From the contract prices paid on this work, which were $42.00 per 1000 ft. B. M. of timber, $10.00 per cubic yard for concrete in crib, and $15.00 for concrete in caisson, 5 cts. per lb. for iron, and 20 cts. per cubic foot of excavation sinking caissons, the cost of the work below water would be $15.06 per cubic yard. The actual cost, taking in consideration accidents, delays, and loss of material, was considerably greater.

* It is but justice to say that Mr. H. F. Lofland, the Div. Engineer in charge of the bridge, earnestly pressed the importance of flooding the caisson in time to have saved it, but unwise counsels prevailed, and it was not flooded until too late—a valuable but expensive experience.

Article LI.

COMBINED OPEN-CRIB AND PNEUMATIC CAISSON.

48. As was mentioned, the writer designed a combined structure for the purpose of reaching rapidly, economically, and certainly a depth beyond that at which the pneumatic caisson can be sunk, upon which he secured a patent. This structure will now be described, both on account of its being in its general design typical of both the general construction of a timber or iron caisson, and of the novel features making it available for use as a pneumatic caisson, or an open crib.

The description will be better understood by referring to Figs. 41 and 41 (*a*), 42 (3), and 42 (4). As a crib, the description already given will suffice (see paragraphs 2, 3, 4, Art. 47, Part Third).

As a caisson it may be stated in general terms that there are one or more decks or roofs, converting that portion of the crib below into a caisson; these roofs are removable in part or entirely. As many separate and distinct air-locks as may be desired or required can be introduced. An iron shaft can be extended throughout the entire height of the crib; any part of this shaft or its entire length can be converted into an air-lock. Piles of 50 ft. or more can be introduced into the air chamber and driven below the lower edge of the caisson. The general advantages secured are that, 1st. To the depth of a hundred feet, or whatever may be the limit of the pneumatic process, we secure the advantages attaching to this process. 2d. That below this depth the structure can be used as an open crib, sunk by the usual methods, securing a minimum vertical lift of the dredged material,—a largely reduced frictional resistance on the outside surface, thereby enabling greater depths to be reached than in any other manner more rapidly and at less cost. And, 3d, should for any reason the crib be stopped by any obstruction, long piles can be introduced and driven until a satisfactory bearing is obtained. 4th. It is specially applicable in very great depths of water where the bed of the stream has not bearing resistance sufficient to build upon it, and where the excessive lift of the dredged material would greatly increase the cost of construction. 5th. It provides those conditions and means of, to a great extent, removing the injurious effects resulting from working in compressed air, adding to the comfort and health of the men, without obstructing or delaying the prosecution of the work, and adding but little to the cost of the structure itself. 6th. For small depths, after sinking as a pneumatic caisson, the roof can be removed, after partly or entirely filling the air-chamber with concrete, by which a solid and uniform mass of concrete or masonry can be built from bottom to top of the piers.

49. Fig. 41 is a vertical cross-section, showing double walls of crib and cross-walls A, which are to be filled with gravel or concrete, which furnishes the weight necessary to sink the caisson and also forming a part of the permanent foundation. As seen in the drawing, V-shaped cutting-edges are formed and built solid for a height of about 9 ft., through which both screw and drift bolts are passed, and all of the walls tied together with long iron rods. From that height the several walls are formed by 12 × 12 in. timbers laid on top of each other and drift-bolted; cross-braces at intervals tie the walls together. These walls are built up as the caisson sinks; the extreme outside walls are built with a gentle batter, or the lower section alone may have a batter, and all above vertical; this latter is common, especially when iron is used. The partitions *C* from wall to wall constitute the various roofs, dividing the space between the walls into a number of chambers 8 or 9 ft. high, marked B in the drawing. The entire outside and also the roofs are calked or otherwise made air-tight. Air-locks *D* built into the roofs afford means of passing from chamber to chamber. In the middle space an iron shaft extends from top to bottom, any part of which or the entire shaft can be converted into an air-lock. These shafts communicate by side-doors with the chambers. An air-pressure due to the depth can be maintained in the chambers, the difference of pressure in the successive chambers being that due to the height of the chambers. Any number of roofs may be used.

The roofs in the middle space are constructed with iron beams and plates riveted to them; those in the two outside spaces are shown with a timber construction. Also air-locks D' afford a communication from the chambers to the spaces between the walls, an open vertical shaft being left in the concrete; the men having the choice of entering or leaving the caisson by this avenue, this twofold avenue increasing the chances of escape in case of accidents. The usual air, water, and discharge pipes, P, are shown. The drawings show some of the air-locks in section, others in elevation. The doors are shown both while open and closed. Fig. 41 (a) is a horizontal section showing the roofs partly removed; B, chambers; C, roofs; D, air-locks and shafts. As the roof may be formed of iron beams and plates, the roofs can be opened by removing the plates, leaving the girders to serve as braces; or, as shown at T, the plates under two of the girders can be left in place, thereby forming troughs into which the dredged material can be emptied, and discharged by the air through pipes. The proper spaces are shown partly filled with concrete in both drawings. Figs. 3 and 4 are part sections, the first showing the method of introducing piles into the caisson through long air-locks and at the bottom piles driven and partly filled over with concrete. Fig. 4 shows the caisson sunk below the limit of the pneumatic process, in which the lower roof C' has been removed except as to necessary bracing; this roof just passing below the water surface, the roof C is as yet intact.

OBJECT AND USES OF THE ABOVE STRUCTURE.

50. Assuming a depth of water, say 100 ft., underlaid by a soft, silty material, into which piles can be easily driven thereby securing a sufficient support. A caisson of this kind could be sunk resting on the bed of the river. Piles could then be introduced after the air-pressure was established, as shown in Fig. 3, and driven to the required resistance, cut off squared, capped if desired, and then concrete built over them to any desired height, and the masonry then commenced. The masonry, if desired, could be commenced on top of the piles.

This evidently furnishes an economical mode of securing a foundation where the depth of the water is great and the underlying material uncertain.

The crib resting on the bottom at, above, or below the limit of the pneumatic process, with the roof C' at this level, the roof C could be partly removed, leaving the trough-shaped braces in place; the material, dredged and lifted into this trough, could be discharged, either by the air-pressure or mud pump, through proper discharge pipes. As the caisson sinks, the roof C reaching the water surface, the roof C_1 could then be partly removed; the men using this as a platform from which to work. BB_2 then being the work chamber, B_2 passing below the water surface gradually. When C_1 reaches the limit, the men ascend to C_2, and so on. These operations are indicated in Fig. 4. In this manner we make use of the pneumatic pressure as far as practicable. We limit the lift of the dredged material to a minimum, and secure the advantage of the rapid and economical methods of removing the material adopted in ordinary caissons. The water is kept at a constant level, the men ascending as the caisson sinks. In addition, the air escaping under the cutting-edges and rising along the sides reduces materially the resistance of friction by loosening the material. There can be no doubt that this process is reliable, expeditious, and economical, and can be used where other means would fail. If the depth should be 200 ft. below the water surface, say 70 feet water and 130 ft. solid material, sink the caisson by the pneumatic process 100 ft. At this point the dredging would commence, the lift gradually increasing from 0 to 100 ft., or an average lift of 50 ft., the air or the pump doing the balance. In the open-crib process the dredging would commence, when the caisson rested on the bed of the river, the first height of lift being 70 ft., gradually increasing to 200 ft., the average lift being 135 ft. It is perfectly evident that this method must be slower, more expensive, and more uncertain.

51. The construction is by no means a bad one for a caisson to be sunk less than 100 ft., or for an ordinary caisson

It would not require, before commencing to sink the caisson, the delay necessarily caused by the time required to construct the ordinary caisson proper. The heavy mass of timber required in the roof would, to a large extent, be avoided. Only one roof would be necessary in this case; but it would be advisable to use at least two, the chamber between being used for a dressing and warming room for the caisson men, through which they could pass as leisurely and as comfortably as may be desired, without obstructing in any manner the progress of the work. It is evident that the roofs should be of iron, as it can be more conveniently constructed and removed. It has the further advantage that, in case it should be found necessary after sinking the caisson to go beyond the pneumatic limit, additional roofs could be constructed and the sinking continued, or piles introduced and driven; whereas, in the pneumatic caisson proper, when its limit is reached, it can neither be sunk further nor removed; and it is possible that, under such circumstances, the structure would be useless, its entire cost thrown away, or an uncertain foundation used.

52. The entire structure can be built either of iron or wood, the choice being mainly one of cost, as the strength in either case is sufficient, or the part below the bed of the river could be wood and the part above of iron—especially if in sea water, where the timber would be destroyed by worms, and also where obstruction to the current or navigation is a matter of moment, somewhat less space would be occupied by the iron wall. In short, the writer does not hesitate to say that it is a good design for any kind of foundation below water for any depth of water or solid material from 30 to 200 ft. It has, however, its special application in those rivers, such as the Mississippi, at New Orleans, where there is a great depth of water, and where any such obstruction to the channel would be bitterly opposed, as the structure could be narrowed to a minimum thickness at any desired depth below the water surface, without in any manner interfering with the prosecution of the work below. "It is claimed in the patent that greater depths can be reached than by any other known method, and

at any depth the work can be done relatively more rapidly, more economically, and more certainly, and that for such depths as require only the ordinary coffer-dam absolute security against breaks and leaks can be secured, and foundations can be constructed either under a moderate pressure, or after fully bracing and sealing up the cutting-edge the roof can be removed, and the work proceeded with safely and securely, as in an ordinary coffer-dam in the open air." See Figs. 41 and 42.

In the year 1889 the writer showed Mr. E. L. Corthell, an engineer of great ability and experience, the plans and descriptions. He was then working on the Memphis Bridge Plans. It had been supposed by many that the depths required for that bridge would be much greater than were afterward found necessary.

In 1890 Mr. Corthell made a report on building a bridge across the Mississippi, near New Orleans, and in this report he recommended the above plan.

GENERAL REMARKS.

53. In all of the foregoing subjects the writer has described, in general terms, the actual methods of the construction of caissons, cribs, and coffer-dams, etc., as practised by himself and many other engineers, and also the subsequent operations of sinking, with more or less detail, without criticism of the methods of others. He has, however, often alluded to the importance of avoiding, as far as practicable, the adoption of what seemed to be useless refinement in the sizes and quantities of materials used in such structures, as well as in the manner of putting the parts together, necessitating increased cost and time required in construction. And in all designs his aim has been to keep in view that good engineering practice only requires that all structures should be constructed in the least possible time, and the least possible cost, consistent with strength, durability, permanency, and suitableness to the end in view. That this does not seem to be the practice of many engineers is apparent in many structures and in many portions of the same structure, and as they do not generally result in

any better work and only add to the time and cost, such practice can only be considered useless and wasteful of both time and money. Attention to some extent was called to this subject in discussing the subjects of concrete and masonry, and the effort was there made to show in what manner first-class work in every respect could be secured without useless and onerous requirements such as are often imposed. Attention will now be directed to similar requirements often imposed in the construction of some deep and difficult foundations.

It is not uncommon to see described in books for the construction of the sides of open caissons, which are simply timber coffer-dams, that they should be composed of planed and tongue and grooved timbers, sometimes of specially large cross-sections, where as timber as it comes from the mill is in every respect as good, no planing being necessary, except possibly planing slightly the edges of the outside plank for a calking joint.

The guide-piles of coffer-dams are often required to be sawed square. Round piles are equally as good, cost less, and can be driven much more satisfactorily. In framing cribs that are to be filled with concrete, it is far better to use round logs for the cross braces, any slight variation in the diameter at the two ends being a matter of little or no moment, and they admit packing under and around them to much greater advantage. And in many cases the entire crib could be constructed of round logs without in any way impairing the usefulness of the structure, as for many purposes under water sap wood is as serviceable as heart wood.

In the construction of the pneumatic caisson particularly there seems to be no regard paid, as a rule, either as to the cost or to the relative strength of the parts; bolts and rods are inserted in large quantities where there would seem to be little or no use for them; and no special attention is given to a strong and rigid connection between the walls of the working chamber and deck of the caisson, which is matter of the greatest importance. For instance, in the deck of the caisson composed of eight or ten courses of timber crossing each other.

drift-bolts 1 in. × 22 ins. are driven at every intersection; this would require in any ordinary-sized caisson some 32,000 drift-bolts, or about 190,000 lbs., costing some $8000 to $10,000, when one fourth to one fifth of these quantities would be ample under any circumstances; and in addition long bolts 1½ to 2 ins. in diameter and 8 or 10 ft. long are put through the entire deck with a reckless profusion, and only to hold timbers together that have little tendency to separate; and similarly in other parts, except that comparatively few bolts are used to connect the deck to the walls of the air-chamber, where the danger really exists, and where the framing is usually such that, outside of the interior bracing, the bolts are the only connections. Often, also, in constructing caissons, all of the timbers are run through a planer, so as to gauge them to exactly the same size. Surely nothing is gained by this; the cost, however, is greatly increased. Unless the timber is badly sawed, an equally good, if not better, work is secured by bedding the timber in cement mortar, and filling the vertical joints with grout. In regard to incasing the cutting edge of a caisson in iron plating, there is much difference of opinion and practice. It can safely be said that it is not necessary; it may be a safe precaution, and it may or may not add materially to the cost. It is claimed by some engineers as a decided disadvantage. The writer has never used it.

Often expensive stagings and platforms are erected to regulate and control the sinking of caissons: here again the writer cannot speak from experience; they will certainly be very costly, and their utility is certainly doubtful. The writer only used a few clusters of piles, mainly to hold the caisson while floating, and to aid in locating the caisson accurately on the bottom, no material error in position resulting during the sinking. Tendencies to move gradually in one direction are sometimes developed, which can generally be checked either by blowing the material to that side, or by settling the caisson slightly out of level, and then levelling it again; reasonable care and watchfulness will ordinarily prevent any trouble. Many such matters do not, of course, admit of any close calculation, and for this reason it is the aim to be always on the safe side, which is commendable so far as it applies; but there is nothing gained by enormously strengthening some parts of a structure and leaving other parts proportionately weak. The writer's object is only to call attention to some of the evidently useless waste of material, money, and time, without any reasonably compensating advantages. Spare no time or money in strengthening weak points, but do not waste them on those essentially strong points, that can take care of themselves.

The writer made estimates for contractors proposing to build a large caisson and sink the same under specifications which fully illustrates the above remarks. A few extracts will be given for the corner-posts:

4	pieces of white oak,	24 ins. × 24 ins. × 16 feet.
4	" " "	16 ins. × 27 ins. × 16 feet.
312	" " "	12 ins. × 16 ins. × 16 feet.

The other timbers for the caisson were of almost all conceivable dimensions: 12 in. × 12 in. × 20 feet, 12 in. × 12 in. × 10 ft. 2 in., 8 in. × 12 in. × 17 feet, and so on; in all 908,616 ft. B. M., every stick of which had to be planed. The writer does not hesitate to assert that an equally strong, durable, and rigid structure could have been built with no variations in dimensions from 12 ins. × 12 ins., except in the lengths of certain parts which have necessarily to be specified; and further, that the planing of the timbers was absolutely without necessity or even advantage. Pine timbers would have been equally suitable. Such requirements simply mean an enormous waste of money and time. In addition, screw-bolts amounting to 43,000 lbs., in all lengths from 2 to 12½ ft., and from 1 to 2½ ins. diameter, were stuck in all conceivable places through the deck of the caissons, through corner-posts, etc., and in addition 1992 drift-bolts 1⅛ in. diameter and 58,348 drift-bolts 1 in. diameter, amounting in the aggregate to 350,623 lbs.; these were used at every intersection in the deck, that is, one foot apart in each direction over each of the seven crosses of solid timber in the deck. Whereas one fifth of the entire number of bolts would have been ample, and the long screw-bolts $2\frac{1}{2}$ in. diameter, and $12\frac{1}{2}$ ft. long, as well as a number of the other sizes, could have been entirely omitted, with a saving of thousands of dollars. The writer suggested these changes to the chief engineer, stating the useless labor and expense involved, only to receive the reply that the work was to be done rigidly according to specifications, and that the company could pay for it. The result was that the lowest bid was over $230,000, whereas with reasonable requirements the work could have been done under $200,000. All bids were rejected; the company undertook the work. Whether changes were made or whether the cost was more or less is unknown. This is but a sample of the reckless waste of money in designing and constructing many works that have come under the writer's observation, and is introduced to show the importance of designing structures with some regard to the relative strength of the parts connected and the connections themselves.

It will be noticed that in this structure there is 380 lbs. of iron to every 1000 ft. B. M. of timber. In the caissons of the Susquehanna River Bridge the average iron in bolts in each caisson ranged from 136 to 152 lbs. per 1000 ft. B. M., being probably more in proportion on the smaller caissons, as many straps, bolts, etc., were of the same dimensions in all cases; this proved ample in sinking through both sand and gravel and silt, and in one caisson a sudden sinking 7 or more feet and landing hard on rock, crushing off the lower end of the verticals and careening at a considerable angle with a heavy load on top, did not spring a leak in the timber-work at any point. On the Cairo Bridge, from the data before the writer, the iron is 414 lbs. per 1000 ft. B. M. This evidently includes the shafts, pipes, etc., as the amount of iron is merely given as so much weight supported by the foundation-bed, and as both the roof of the caisson and the high cribs (34 ft.) were open-built, there was relatively a smaller proportion of timbers and a larger proportion of concrete, necessitating a larger ratio between the iron and timbers, though the actual quantity of iron in pounds was small.

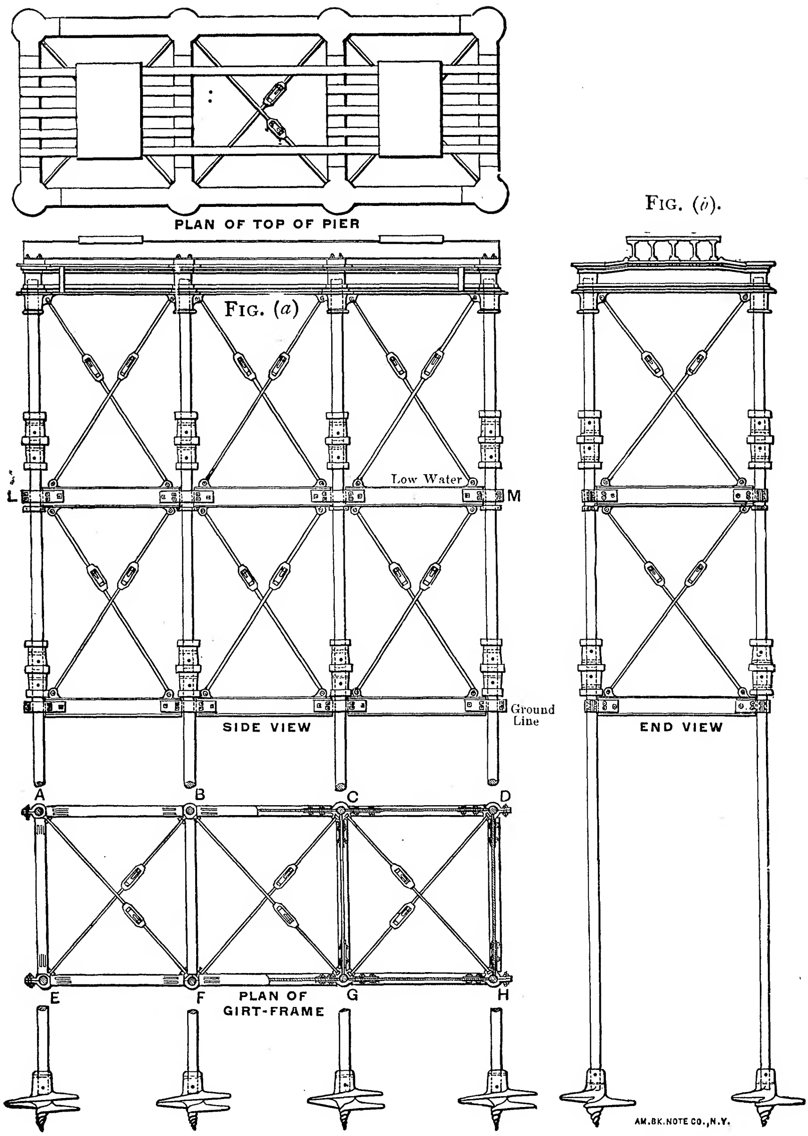


FIG. 54.—SCREW-PILE PIER, MOBILE RIVER, L. & N. RY.

[*To face page* 321.]

Article LII.

ALL-IRON PIERS.

54. Iron piers can be constructed with either cast or wrought iron columns. The wrought-iron columns are composed of latticed channels; several such columns being placed in slightly inclined positions, these are braced with horizontal channels or other form of struts, and diagonal tension members between them. In rivers liable to great rises, bringing large masses of driftwood, these piers should be incased in plate-iron; this is generally open work, consisting of flat strips placed at intervals, or large lattice strips; this while not entirely opposing the current turns aside the drift and prevents large masses collecting or getting tangled up in the braces. Such piers are light and should be strongly anchored to low masonry piers, the piers being built up to or a few feet above low-water, and in very high bridges up to or above high-water. Two good examples of such piers can be seen,—one across the Alabama River, near Montgomery, and a second in a bridge recently constructed across the James River, near Richmond, Va. There are some serious objections to such piers, unless the masonry is carried up above high-water, in rivers carrying much drift or large masses of ice, and they are not common in such cases.

55. A description of an all-iron screw-pile pier bridge, constructed by the writer across the Mobile River, about 16 miles above Mobile, Ala., will be interesting and instructive. The total width of the Mobile River at this point was about 1000 ft. This distance was divided into seven spans by six screw-pile piers and two brick abutments resting on ordinary piles. There was one draw-span 260 ft. from end to end, giving two clear openings of about 112 ft. each.

It may be as well to mention that screw-piles may be of wood or iron, solid or hollow, varying in diameter from 6 to 12 ins. or more, having a screw-disk at one end, similar to one turn of an auger, which may be from 12 ins. to 6 ft. in diameter. They are screwed into the soil, soft rock, coral reef, etc. Hand or steam power can be used. For ordinary piers there are from 6 to 8 piles to the pier. The bearing surface being the sum of the areas of the screw-disk, the friction of the material on the surface of the shaft will add something to their bearing capacity.

56. In each of the screw-pile piers in the Mobile River there were 8 solid wrought-iron shafts, diameter of screw-disks 4 ft. The pivot pier was composed of one centre shaft 8 ins. diameter, and 10 shafts of 6 in. diameter distributed around the circumference of a circle about 25 ft. diameter. The screw-disk for the centre shaft was 6 ft. diameter. All other shafts were 6 in. diameter with cast-iron screw-disks 4 ft. diameter. The rectangular piers were formed in two rows, 4 piles to each row. The piles in each row were 8 ft. centres; the rows themselves were 9 ft. centres. The piles were braced by eye-beam struts connected around the shafts by collars which were bolted to the beams, and diagonal tension rods in both vertical and horizontal planes. The piles were capped with heavy cast-iron pieces bolted together through flanges, short wrought beams resting lengthwise of the pier on the caps, and on these a thick iron bridge-seat. All parts well bolted together. The eye-beam struts with the horizontal diagonal rods were called girt frames. Three or more of these were used according to the height of the pier above the bed of the river. Drawings and full details of these piers are shown in Fig. 54 and Fig. 54, (*a*), (*b*).

57. The piles of the pivot pier were braced by eye-beams between the piles and radiating from the centre, and a system of diagonal tension rods in various inclinations. These piles were also capped and connected together on top by large cast-iron pieces bolted together, upon which was placed the necessary turn table arrangements.

The general lengths of the spans were about 142 ft. The design of the superstructure was the well-known but little used post truss, in which both the tension and strut web members are inclined. The original contract contemplated screwing the piles to a depth of 45 ft. below the bed of the river; the actual result was that the greatest penetration below the bed of the river was $18\frac{1}{2}$ ft., and the average $15\frac{1}{2}$ ft. and then in one or more cases the shaft commenced to twist, and in all cases the steel teeth, presently to be explained, cut iron shavings from the pile, without turning or screwing them into the material, which was a fine compact sand.

These indications were accepted as satisfactory proof that sufficient bearing power had been secured. One idea in adopting this kind of pier was to prevent scouring by offering as little obstruction as practicable to the current, and thereby prevent any scouring. These piers are light but strong and stiff, and have now been in use over twenty years, and carry safely the heavy rolling loads of the present day.

58. Some little detail concerning the manner of sinking them will be given. The depth of water varied from 10 to 20 feet; the height of pier above water, about 9 feet. At the site of the bridge the rise from floods was only a few feet; the immense volumes of water from the rivers above dividing among many bayous, and spreading over the entire swamp.

The shafts were rolled in sections of different lengths; the bottom section, which was connected with the screw-disks by four steel pins, was about 22 feet long. This, when set in place, would reach above the water; on top of this a heavy cast-iron sleeve about 3 feet long, fitting snugly around the pile, and fastened to it by two steel pins at right angles. Another section 12 or 15 feet long was then lowered into the sleeve, and resting on the top of the first shaft. Steel pins were then passed through sleeve and shaft. The machine for turning the piles consisted of a rectangular base frame of timber, to the corners of which were fastened four stout pieces of timber meeting at a point above, which was slightly out of centre in one direction, so that the shaft when standing vertical and in position would clear the timbers of the frame. About four feet from the bottom of the frame a large cog-wheel, supported horizontally, was placed; the spokes of this wheel rested in an iron collar about 15 ins. diameter, carrying on the inside two friction-rollers. The shaft passed through the centre of the collar. A strong jacket with flanges in two halves, and carrying on the interior several solid steel plugs with sharp teeth, was adjusted to the shaft and drawn by bolts close to it, indenting the teeth into it by bolts through the jacket-flanges, the lower ends of which rested against the friction-rollers; a worm-screw with ordinary crank-arms could be thrown in or out of gear with the cog-wheel. A number of men turning the worm by its arms imparted a circular motion to the cog-wheel, which turned the jacket and the embraced shaft, thereby screwing the disks into the bed of the river. This could be continued until the top of the jacket reached the rollers; the jacket was then loosened and lifted to a distance equal to its length, again tightened to the shaft, when the power could again be applied. From 8 to 10 men could apply power enough to twist the shaft. The use of steam might have been more economical and rapid, but would not have been more efficient. The greatest difficulty existed in starting and holding each pile vertical and in its exact position; this was essential, as otherwise the caps and girt-frames could not have been adjusted to the piles, as all parts were made in Chicago and shipped to the bridge-site. The work was carefully and conscientiously executed by Gen. Wm. Sooy Smith. In addition, every pile had to be brought to exactly the same level on top, as it would have been very troublesome and expensive to cut them to a level. This was accomplished in the case of every pile, except the large centre-pile of the pivot pier, which could only be screwed 9 feet into the sand; this was clipped off with the cold-chisel. The greatest error in the levels of the top did not exceed one eight of an inch.

When from any cause it is necessary to reach greater depths than the piles can be screwed by turning, the limit of which is reached when the piles show signs of twisting, or the teeth or other hold upon the pile is insufficient, resort can be had to the water-jet. This method has been used successfully. It is stated, no doubt, upon reliable authority that the use of

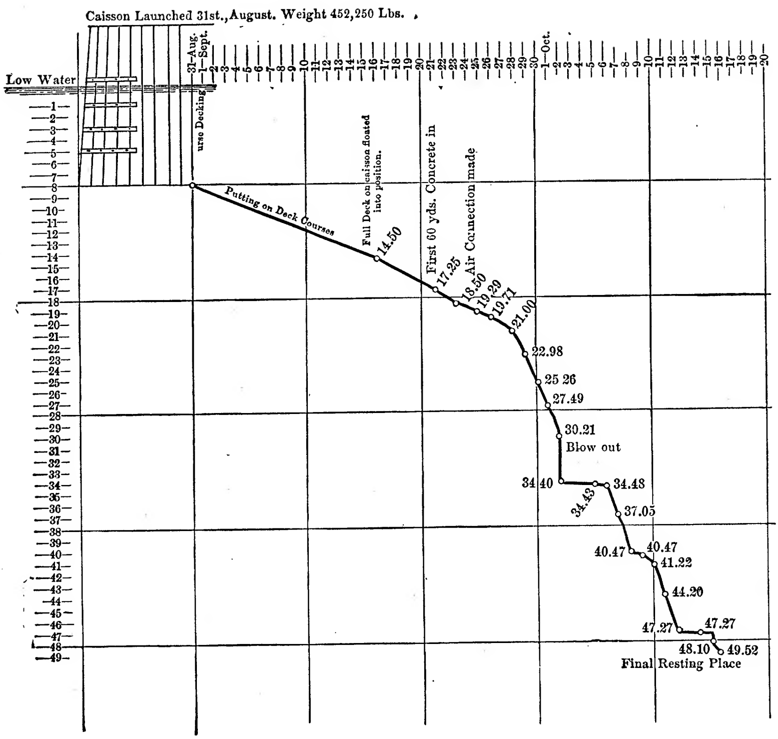


FIG. 55.—SHOWING DAILY PROGRESS IN SINKING OCTAGONAL CAISSON.

[*To face page* 324.]

the jet is more effective when applied to the upper surface of the screw-disc, rather than, as would seem natural, to the under side and the point of the pile. Why so, does not seem entirely clear. Applied to both under and upper surfaces would, no doubt, be advantageous. In this process there would seem to be no cause of trouble when the screw-disc was not over from 12 to 18 inches in diameter; but with discs from 3 to 6 feet in diameter it would be troublesome to hold a pile exactly vertical and in exact position, if this should be absolutely necessary, as in the case already mentioned. On this point the writer, however, cannot express an opinion, as he has no experience in this method of sinking screw-piles. Each pile in each pier had to be located separately from an established base on the shore or from completed piers, as no staging could have been constructed steady enough to maintain any centre point.

Article LIII.

LOCATION OF PIERS.

59. There are many methods of locating the piers of large bridges across rivers. They all, however, resolve themselves into the method of triangulation, or direct measurement from some established base on the shore; and as it all depends then on the base-line, this should be accurately measured, and its direction and location in regard to the centre-line of the bridge should be carefully selected. It should be as nearly at right angles to the centre-line as practicable; and its length should be equal, or nearly equal, to the entire width of the river, so that distances from the end of the base, equal to that of each pier from the same point, can be laid out on the base-line. It is, however, rare that both of these conditions can be realized in practice; especially as it is also desirable that the base-line should be laid out on ground as nearly level as practicable. This, however, is not a matter of so much importance, as with due care perfectly accurate distances can be measured on rolling or rough ground. But it is essential that each pier shall be easily visible from its own triangulation points, and that the entire base shall be seen from either end. The best adjustment of the base to all of these conditions must be made. No angle in a triangle should be less than 30 degrees, nor greater than 120 degrees. The base may be somewhat less in length than the width of the river. It is advisable to have a base on both sides of the river, the one used as a check on the other; 2d. If points can be found, two on each side of the river, so that the lines joining two of them is near to, and approximately parallel to, the centre-line of the bridge, and so situated that each and every pier can be seen from both extremities of each line, these lines form an excellent basis, and are good checks on each other. The lengths of these lines have to be determined from bases, which form well-conditioned triangles with them; but otherwise selected without reference to the centre-line of the bridge or the positions of the piers. If the two secondary bases across the river are in sight of each other the one can be used to calculate the length of the other, thus insuring the accuracy of both. These lines should be far enough from the centre-line so that the directions of each pier from its extremities shall form well-conditioned triangles with the base; 3d. Or bases can be measured on opposite sides of the river, extending in opposite directions—one up and one down stream. Upon these lines points can be established, so that the lines joining two of the points shall intersect the centre-line at the centre of each pier. This method has the advantage that when these points are once accurately located it is not necessary to turn any angles to locate the position of a pier, as it depends upon the intersection of two lines ranged by foresights, and the further advantage that the engineers are working from largers to smallers, and any error in centring the rods with the transits are divided or lessened, eliminating two sources of error: that of working from smallers to largers, if the one base is too short, and the error of graduation in the limb of the transit, as well as the error in reading the vernier. In addition, this

FIG. 56 (*d*). BASE-LINE BAR.

method only requires the measurement of one angle for each base, viz., the angle between the base-lines and the centre-line of the bridge; the base-lines need not be parallel. The distances from ends of the base-line to each pier, measured on the centre-line, must be known. Having determined the width of the river between points established on the shore on the centre-line, and the position of the piers on this line, the piers can be located by either of the three methods. Fig. 56, (*a*), (*b*),

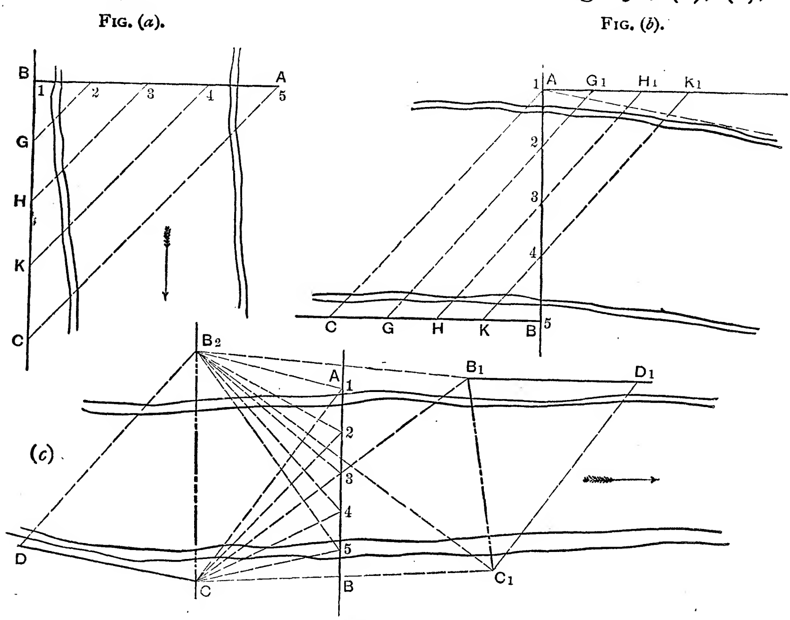


FIG. 56.
METHODS OF LOCATING POSITION OF PIERS FROM BASE-LINES.

and (*c*) show these methods in their order as above described —in which *AB* is the centre-line; *BC* the base-lines, from which the piers are to be located; 1, 2, 3, 4, and 5 the positions of the piers. The base-lines in Fig. 56, (*a*) and (*b*), are shown as passing through the centres of the shore piers, and the triangulation points at the centres of the piers on the other side of the river. These may occupy any position with respect to the piers that may be found most convenient. In Fig. (*a*) the angle at *B* is known, and also the distances *BG*, *BH*, *BK*, and *B*2, *B*3, *B*4, and *B*5, and the angles at G, H, and K calculated. In Fig. 56, (c) the bases B_2C B_1C_1 are calculated from the measured bases B_1, D_1, and DC respectively, the accuracy of which can be tested by calculating B_1C_1 from B_2C used as a base. The distance from 1 and 5 piers can then be calculated from either B_2C or B_1C_1 as a base—these piers being located on the banks of the stream, as may be determined by purely practical considerations; the main object being to place them far enough back from the sloping banks to preclude any danger from caving in of the banks. The angles B_22C and $C2B_2$ are then easily calculated in the triangle $1B_22$, AB_2, 1.2, and the angle $B_21.2$ are known, from which $1B_22$ can be calculated; then $2B_2C = 1.B_2C - 1.B_22$, and similarly for all other required angles. As seen DC and B_1D_1 can be measured where convenient without reference to the centre-line AB. In Fig. 10, having measured the base BC, lay off the distances BK, BH, BG, and BC, approximately equal to $B4$, $B3$, $B2$, and $B1$, respectively. The proper distances AG_1, AH_1, and AK_1 can be easily calculated. For instance, we know in the triangle $B2G$, BG, $B2$, and the angle $GB2$, then angle $G2B$ can be calculated. Then in the triangle $A2G_1$ we know the angle $A2G_1 = G2B$, the angle $2AG_1$, and the distance $A2$, from which calculate AG_1. In every case two transits are required to locate the position of the piers; one of them at least should be of first-class make, with a good telescope and accurate limb graduations. Several points in the prolongation of AB on each side of the river should be established; and large hubs of good solid wood, from $2\frac{1}{2}$ to 3 feet long, should mark these points—the exact point marked by a tack. The top of these hubs should be even with the surface of the ground, or better, a few inches below, to prevent its being disturbed by hauling over or near it. The centre-line should be well and distinctly marked on the faces of the piers before they rise above the line of site. The intersection of all the oblique lines, with the faces of piers, should be marked also. A line of red or black paint answers for this purpose; and on the completion of the pier its exact centre, both as to distance and line, should be marked by two chisel-scratches intersecting and painted, or by drilling a small hole and inserting a short iron rod. In measuring the base-lines large hubs should be driven not over 12 to 15 feet apart, accurately lined, on level ground; these should then be sawed off square to the same level. On rolling ground as many should be cut off to the same level as practicable; and any change in the level required is to be made at one point, and then cut off as many as practicable at the new level. The base can then be measured with an accurate steel tape; driving tacks in line, and at the proper distances apart, to mark the important points. The most satisfactory method is to have made at least three timber base-bars, 12 to 15 feet long; these are made of two pieces of white pine about $1\frac{1}{4}$ in. thick and 3 in. wide; the one set edgewise on, and at right angles to, the other, and bolted together, showing a T-section. These can be lightened by rounding gently from the centre to the ends. Brass strips with pyramidal-shaped ends are then bolted to their ends. Having obtained a 3-ft. standard U. S. steel bar, these bars are accurately measured by them, the brass point being filed to some exact distance, say 15 feet—the brass point not being over $\frac{1}{8}$ in. square; the measurement being made at the standard temperature as nearly as practicable; the sketch (Fig. 56, (*d*)), shows one end of the base-bar ready for use. These three bars should then be placed in line, resting on top of the hubs, with their brass points in contact; the rear one should then be moved to the front and placed in contact with the front one, and so on; the extreme front end being marked on the tack with a scratch to avoid slight errors caused by moving the rods. This should be continued from end to end of base, and repeated several times; then checked by steel tape-measure. In a number of bridges across wide rivers, with high piers placed at all intervals from 100 to 525 feet apart, the writer has never had any appreciable errors in locating the piers.

60. He has, however, relied to a great extent on measurements with steel wire, using generally what is known as No. 10 pianoforte wire. This is very strong and light, can be pulled almost to a horizontal line with a spring-balance; a pull of 15 to 20 lbs. is sufficient. The base-line was measured

carefully, as already described; after which all hubs, except those marking the lengths of the span, should be removed, so that in stretching the wire it will have the same sag that it would have when locating the piers. These hubs were well protected, so that they could not be disturbed. A tarred string will adhere to the wire when tightly wrapped around over a distance of 1 to 1½ ins., the inner edges of the string being at the required distance apart from tack to tack on the base-line. This distance being measured with wooden rods of standard lengths, is independent of the temperature. The wire should be stretched on the base before measuring the span. The contraction or expansion can be allowed for on the spring, when appreciable, without moving the strings; and after measuring the span it should again be tested on the base. This is a safe precaution, but the tarred string never slipped in the writer's experience. A change of temperature of 180° would change the length of a wire 525 ft. long 0.66 of a foot, —about 8 inches, coefficient of expansion taken at .00125. Assuming that the strings were adjusted at a temperature of 60°, then at a temperature of 90° the length would be altered 1⅓ ins., that is, lengthened; and at 30° it would be shortened 1⅓ ins. These would be nearly the extreme ranges of temperature. But this is of no moment, as the wire is tested on the base before measuring. A little greater or less pull on the spring-balance would correct the error. Both the transit and wire should be used to check each other. The transit-rod for this work should be a ⅝ or ¾ in. pipe, brought to a well-defined point at one end, and painted in alternate lengths of a foot red and white.

LOCATION OF BRIDGES.

61. The writer has been often asked what are the considerations determining the location of bridges. The factors entering into this matter are various. 1st. Economy; this involving such questions as the width of the river, the depth of the water, the nature of the material forming the bed of the river, the depth of the foundation-bed below the surface, etc.; the slowness or rapidity of the current. These questions must all be considered and that site selected which costs the least, if economy alone is to be considered. High banks on one or both sides are generally desired, as they decrease the cost of the approaches, though they may increase the cost of the bridge proper. Again, without regard to cost of the bridge proper, the necessary or best location of the line on either side of the bridge may be the controlling consideration. This may or may not be controlled by a question of total cost. A good illustration of this is in the case of the Susquehanna River bridge at Havre de Grace. If the line had been located two miles higher up the river, a bridge could have been constructed resting on solid rock exposed at low-water, instead of building a bridge at a point where we had to go to a depth of 90 ft. below water surface for a foundation-bed. This would have, however, lengthened the line some four or six miles, and would have caused some sharp curves. Six miles extra distance causes much extra cost, both in construction and in maintenance for all time, and means ten or fifteen minutes' more time in running between Baltimore and Philadelphia.

62. Bridges should be easily approached from both directions, avoiding both sharp curves and steep grades. In fact, we are often forced to build at certain points, no choice being left to the engineers, especially in crossing navigable rivers, as permission has to be obtained from the Secretary of War, and, in addition, he determines the lengths of the spans, heights of the piers, as well as site of bridge. The necessities of the case first determine the site. After this economy, considered as applied both to the bridge and the construction of the line on both sides, determines the selection of a bridge site.

63. Economy also demands the height of the piers to be as little above high-water as practicable. On navigable streams this height is regulated ordinarily by law, whether a draw-span is used or not. Likewise, to a large extent, the position of piers, as well as length of span, is determined by law. But when not so regulated, it may be stated as a general rule that, where the foundations are inexpensive, rela-

tively speaking, a number of piers and short spans will be economical. Where the foundations are deep and costly, few piers and long spans are to be preferred. In either case the aim should be to make the total cost as small as possible by many trials with different lengths of span.

64. It is advisable, as far as possible, to avoid bends in the river, as the piers should always be placed with their longer axis parallel to the current; for the same reason, the line should cross the stream at right angles to the direction of the current.

65. The following table gives a few examples of the longest bridges, longest single spans, with the highest piers and lowest foundation-beds now in existence:

	Total Length.	Longest Span.	Nature of Foundation.	Depth Sunk Low-water.	Depth Sunk High-water.
New York Suspension Bridge	5890	1595	Caisson	78	
Poughkeepsie Cantilever	4595	548	Crib	132	
Havre de Grace Truss	6300	525	Caisson	90	94
Memphis Cantilever	7997	790	"	96	131
Hawkesbury Truss	....	416	Crib	153	160
The Forth Cantilever, 2 spans, each	....	1710			
St. Louis Steel Arch	1550	520	Caisson	94	136

The above data are taken, in some cases, from unofficial sources, but are very close approximations, and serve the purpose of showing the depths which can be reached by well-known methods of construction.

Article LIV.

THE POETSCH FREEZING PROCESS.

66. The writer will give a short description of the freezing process, which has been used to a limited extent in sinking very deep shafts, and generally through the most difficult and treacherous material with which the engineer has to deal, namely, quicksand, which always is troublesome and expensive to encounter, and often has opposed an insurmountable barrier to further progress. It has been used in Europe to a considerable extent, but to a very limited extent in this coun-

try. It has been successful where applied, but the public are as yet to a great extent left in ignorance of its relative cost, nor has its possibilities been sufficiently developed to form a definite opinion as to its range of applicability. The owners of this patent are the well-known and reliable firm of Sooy-smith & Co., and it will doubtless be pressed to its full practicable value and usefulness by them. The following brief description is obtained from them and other sources:

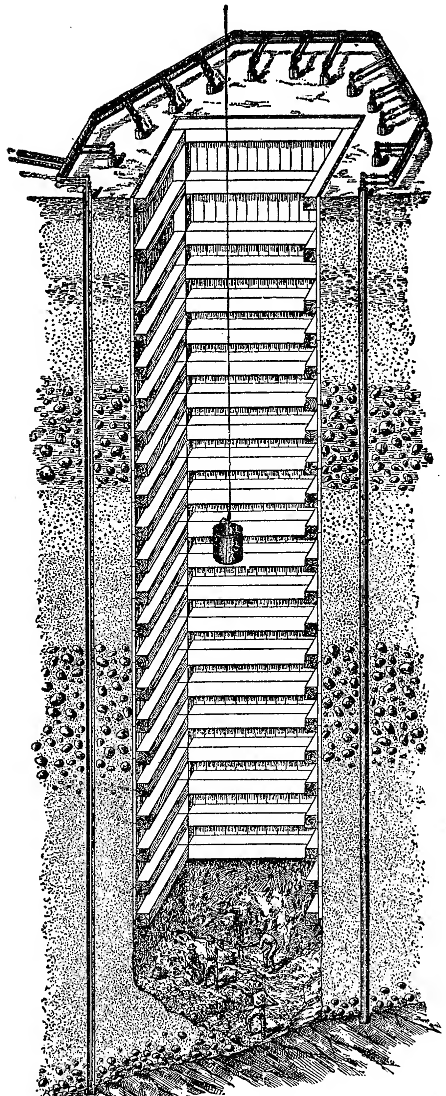
67. A series of vertical pipes 10 ins. in diameter, open at both ends, are sunk around the space to be excavated to rock or some impervious strata. These may be called the pilot pipes. Inside of these, pipes 8 ins. diameter, tightly closed at the lower ends. Inside of these latter pipes, smaller pipes, open at the bottom, are inserted. Each set of pipes, being connected in a series by itself, communicates either directly or indirectly with a cooling tank. The freezing liquid is pumped through the inner small pipes and returns through the outer larger pipes to the cooling tank, to be cooled again and again circulated through the pipes. For convenience and economy these pipes are arranged in a circular form around the space to be excavated. As the cooling mixture circulates it freezes the soil in the form of an increasing solid cylinder or core, which unites at points between the pipes, thus forming a solid frozen wall around the space, the enclosed space being either entirely or partly frozen. The excavation is then commenced, leaving sufficient thickness of frozen wall to resist the outside pressure. For safety, however, in the present development of the process the shafts are, or have been, lined with frames and sheeting as the excavation progressed. The costs then are: first, the cost of the necessary machinery and plants, including pipes and the freezing fluids, etc.; second, the sinking of the pipes to the required depths, and subsequent removal of the same; third, the excavation of the material in its frozen state. This last must necessarily be very expensive, as it is estimated that the crushing resistance of frozen quicksand may be as high as 1000 lbs. per square inch. Lining may not be necessary when the frozen wall is cylindrical, with small diameters; but with large

rectangular piers they would have to be of very great thickness to resist the outside pressure, unless well braced against it. Experience will, however, settle these points, and speculation is of but little profit. Such is the process, and very simple it is.

68. The conductivity of earthy materials either partly or fully saturated with water is not known, and as there is doubtless more or less movement of the water in the water-bearing strata, a sufficient degree of cold must be provided and kept up during the entire time of excavating and lining the shaft.

It is estimated that, as the specific heat of quicksand is only one fifth as much as that of water, the amount of cold necessary to freeze 1 cu. yd. of water would freeze $2\frac{1}{2}$ cu. yd. of quicksand, and that one horse-power per day would freeze 362 lbs. of water.

69. Ordinary refrigerating machines act upon the principle that when a gas is compressed its temperature rises and when it expands its temperature falls. Ammonia, having a high specific heat, is probably the most economical gas to use. "The ammonia may be compressed mechanically, or it may be compressed by the tension of its own vapor heated in a still," which is cooled by passing through coils of pipe immersed in water, retaining its pressure in the still, and when allowed to expand in other coils or pipes its temperature falls rapidly to well below zero. In this condition it absorbs heat from any material with which it comes in contact, by which its own temperature rises. It is then cooled, allowed again to expand, with the ultimate result of freezing the earth or water surrounding the pipes. The efficiency of the now existing machinery is only about twenty-five per cent of the energy applied. The cold gas may be circulated directly through the pipes in contact with the soil, or it may more conveniently be employed to cool a brine, which is then circulated through the pipes. At Iron Mountain, Mich., where a shaft 15 ft. square was sunk to the depth of 100 ft., there were used twenty-seven pipes 8 ins. diameter, arranged on a circumference of 29 ft. in diameter, the pipes being a little over 3 ft. apart. In ten days from start-



Shaft sunk at Iron Mountain, Mich.
BY THE
PŒTSCH-SOOYSMITH FREEZING CO
No. 2 Nassau Street, New York.

FIG. 57.

ing the frozen cylinders were in contact. From this time the enclosed space froze more rapidly than outside the pipes, for obvious reasons. Strata containing little water were frozen to a greater distance from the pipes than those containing much water. "An ammonia machine of the compression type (the ammonia compressed mechanically) was used. Its capacity was twenty-five tons of ice, or fifty tons refrigerating capacity, per day. The wall was frozen and the excavation to the ledge of rock (100 ft. down) was completed in two and a half months from the time that the ice machine first started." The circulating brine was calcium chloride, on account of its low freezing-point, high specific heat, and non-corroding action on iron pipes. "The best results are obtained from such a rate of circulation that there is but little difference in temperature between the outgoing and incoming brine. A very efficient temperature for the outgoing brine is 10° F. below zero, and pumped at such a rate that the return flow is 2° higher." The subject is a very interesting one, and it remains as yet to be determined, the cost as compared with other methods at the same depth, its certainty as against leaks, breaking in of walls at great depths, the relative time taken to complete structures requiring such large bases as the piers of bridges; and, until applied on such large scales which may develop either unknown difficulties or advantages in the process, it would be unjust to the owners and to the engineering profession alike to forebode either evil or good concerning it, and it is to be hoped that its owners may be bold enough to make the experiment on a large scale. The drawing, Fig. 57, page 335, shows positions and arrangement of pipes, the excavation made through sand, gravel and bowlders to rock, and the timber lining for shaft.

ARTICLE LV.

QUICKSAND.

70. HAVING now described the various materials on which structures are more usually built, and the many means adopted to secure a safe bearing for both shallow and deep foundations, a few facts in connection with the nature of, and difficulties to be encountered in dealing with, the most troublesome, treacherous, and almost unmanageable material, namely, quicksand, will be interesting and instructive.

It is not uncommon to consider as quicksand any kind of material, so saturated with water, that it will flow more or less freely when its natural condition of equilibrium is destroyed, by excavating pits, trenches, shafts, or tunnels. This material may be found on the surface underlaid by a firm material, or it may be found in strata of greater or less thicknesses confined by firm strata both above and below. When on the surface, though presenting some difficulties, it can be dealt with by any of the methods heretofore described, and will not therefore be further discussed here. The most troublesome case arises when strata of quicksand are met with at considerable depths below the surface. The reasons are many and evident: the pressure is likely to be much greater; the flow of the material allows the superincumbent strata to settle, bringing an almost irresistible pressure upon the sides of the structure, either crushing it in or at any rate throwing it out of line, increasing greatly the amount of material to be excavated; these causes adding enormously to the cost of the structure and time required to complete it. Sometimes the crude methods of overcoming these difficulties, regardless of delay and cost, such as the free use of straw, brush, shavings, extra sheeting and bracing, have proved successful, but often, after repeated efforts, and expenditure of money, the further prosecution of the work had to be abandoned. The discovery and application of the freezing process was a source of hope and encouragement, and that it is effective cannot now be questioned or denied. This and the introduction of a new method, presently to be explained, requires a more accurate understanding and definition of the loose material called quicksand.

71. In a pamphlet written by Mr. E. L. Abbott, dated Nov. 20, 1889, on the freezing process, and doubtless having the sanction of such high authority as the Sooysmith Company, quicksand is defined as any earth which "will in some

degree run like a fluid when mixed with water." He, however, states that any kind of sand mixed with a small amount of clay possesses this property, but that the most troublesome material contains but a small per cent of very fine sand. "*This material when undisturbed may have some consistency*" (italics mine); when disturbed will flow through any minute opening. In the *Engineering News*, April 28, 1892, in which the new invention of Mr. R. L. Harris is described, is found this statement: "This quicksand, when dry, is an impalpable powder. When saturated with water it is very compact and hard until disturbed. Under the pressure of a slight depth it becomes apparently almost solid; hammer strokes of 300 ft.-lbs. aided by a wash pipe, causing 2-in. iron pipes to penetrate upon an average less than 0.1 in. per blow. Upon being agitated with water the quicksand becomes alive and runs like mush. Its currents under pressure move glacier-like, and are seemingly irresistible."

The writer built some culverts on a quicksand; the solid material composing it was an impalpable powder, and it would run into the excavation like "mush." He also drove piles for a trestle several hundred feet in length through what was called quicksand. No difficulty occurred so far as penetration was concerned, the piles moving several feet at a blow, but immediately after impact the piles would lift the hammer, and removing it, they would spring up suddenly to the height of several feet. There is clearly several kinds of this troublesome material.

72. The freezing process is applicable whether the solid material composing the quicksand is clay, sand, or mixed clay and sand. Time and expense alone are questions to be considered. This method has been explained.

73. The latest method (see *Engineering News*) is novel; experiment proves it effective in the material described. The importance of knowing the nature of the solid material arises from the method adopted, as it depends upon the hardening of injected cement. It is generally accepted that pure cement mixed with clay or mud or exceedingly fine sand * either does not harden at all, or at any rate imperfectly; and in such cases the cement must be "doctored" with sand, plaster of Paris, or anything else that will solidify under the existing conditions. In the case to be described, the work consisted in thus solidifying a trench for a large intercepting sewer, the laying of which had baffled all the efforts of the engineers, and practically bankrupted the contractors.

When such high authority as the *Engineering News* publishes the following: "This [the freezing] process has proved very useful in many cases, but from its very nature it requires a somewhat expensive refrigerating plant, a long-continued circulation of the freezing fluid, and a continuance of the circulation so long as it is desired to keep the material solid, if it is to be exposed for any considerable time."

"The method to be described is the invention of Mr. Robert L. Harris, N. Y. Am. Soc. C.E., and it has been recently tested experimentally on a sufficiently large scale to establish fully its practicability, under proper conditions, in our judgment. It seems likely to prove a competitor of the freezing process in some fields, besides having useful application in cases where that process would not be suitable."

The writer needs no excuse for giving a detailed description of the method.

The principle involved is simple, and depends only upon the fluidity of the material when mixed with water. If two pipes be sunk into the material at distances apart varying with the depth, and a current of water be forced through one of them, it will seek an outward passage along the line of least resistance, and will issue, carrying some of the material from the other pipe along with it, washing a channel between the two pipes, and

* The writer condemned a large quantity of exceedingly fine sand; subsequent tests and experiments satisfied him that the mortar produced was equal to any previously used on the same work with a good-sized and sharp-grain pit-sand. Two large piers were built of this fine sand which stood immersions in flood water, covering it shortly after being mixed and used (the same day), it also stood well a severe winter on exposed surfaces of masonry. It was an almost impalpable powder when dry.

by using a number of pipes a chamber will be scoured out. Channels or chambers being made, "the plan was then to substitute for the channel-making stream of water" some cementing material in a fluid condition, and by proper arrangement of valves shut the outlet pipes as the cementing fluid reached them, and by applying pressure not only make the cementing fluid fill the chamber, but also permeate the adjacent materials, thereby forming a floor between the pipes and by gradually raising the pipes additional layers would be formed uniting into a solid wall. The trench for the sewer was 12 to 16 ft. wide and from 20 to 30 ft. deep in quicksand; the length through this material was over 4000 ft. An attempt to pump out this material was made, and an area of forest about 150 × 75 ft. settled several feet, inclining large trees at a considerable angle and doing other damage. He found that quicksand in solution took hours to settle, but on introducing the cementing material all solid matter settled rapidly, leaving clear water on top; hence, by agitating the material, a large quantity would be placed in suspension, and the introduction of the cementing material would result in an intimate mixture and precipitation of the material, ultimately forming a solid floor. Pipes had been lowered to a depth of 25 ft. and at distances of 4, 10, and 14 ft. apart, which established fully the circulating theory. Then 4 pipes were sunk at the angles of a quadrilateral 4 ft. on the side, and sunk 17 ft. below the excavated surface. A chamber was formed, and after maintaining the cavity for three days, the cementing material was forced through the pipes into the chamber, resulting in a fairly solid and complete floor. Two-inch pipes were first sunk, and a small cavity hollowed out at the bottom. Smaller pipes, carrying suitable valves at the bottom for closing the pipes against upward currents, were lowered through the larger ones. When the ends of the smaller pipes were below those of the larger, the circulation was unobstructed; by slightly raising the smaller pipe, the fluid could not escape. The blocks of cemented quicksand were from 3 to 6 ins. thick, reaching from pipe to pipe; they were hard and solid, and homogeneously solid—in some cases for a thickness of 6 ins. or more. The following figures illustrate the process: Fig. 58 shows the manner of making a solid wall by successive slight lifts of alternate pipes, using first one and then the other for the downward current, and the two adjacent pipes for the upward current. In forming a floor (see Fig. 58 (*a*)), the pipes are simply distributed over the space at regular intervals, and sunk practically to the same depth; the shaded portions representing the solid layers or blocks of

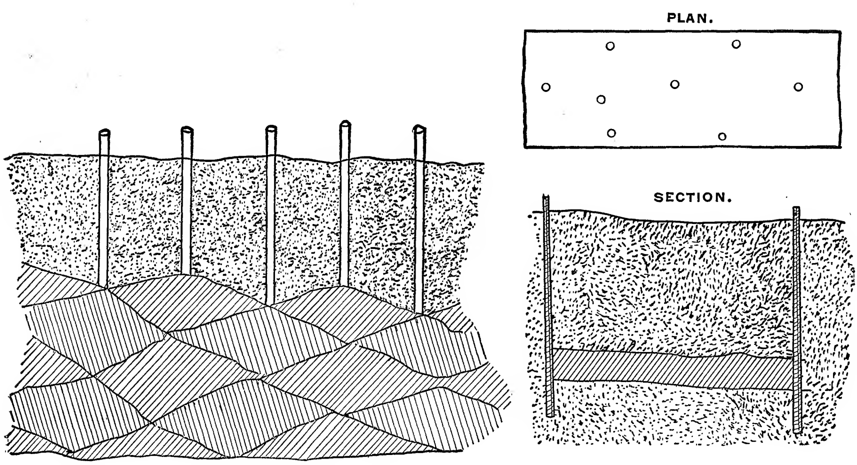


FIG. 58. (*a*).
CEMENTING, THROUGH PIPES, QUICKSAND FOR FOUNDATIONS OR WALLS.

cemented quicksand. Great claims are made for this process in protecting shores, driving shafts, and tunnels, and in putting in foundations forming the base of the materials in place.

The essential principles are to so arrange the pipes as to allow free circulation while washing out, and to close the discharge pipes when the cementing material is forced in. Pipes should not be allowed to be caught in the hardened material. Whatever may be the possibilities of the method, which must be established on a sufficiently large scale by experiment, the process is simple, and seems to be effective. Outside of the pipes the only plant required is a pump of sufficient capacity to produce a good current in a 2-inch pipe, and a moderate pressure when introducing the cementing material.

Both of these methods are patented.

74. In case of the culverts founded on quicksand, the writer simply spread the base by logs crossing each other in several course, and covered with plank upon which the masonry rested. Although this method is not to be recommended, and should not be used at all under very heavy structures, the culverts have, nevertheless, carried safely railway trains for years. Particular care was taken to sheet around the sides and ends so as to confine the quicksand as much as practicable.

75. Hollow brick or concrete or iron cylinders and timber-lined shafts are often sunk to great depths through these soft materials, and ultimately filled with concrete or masonry columns or pillars. The sinking is effected by simply excavating the material from the inside and adding weights, if necessary, to the cylinders sufficient to make them sink against the friction; or by the ordinary method of suspending the upper top setting or frame at the surface, and as the excavation advances placing other strong frames of timber at intervals of 4 or 5 feet, and inserting plank sheeting on the outside resting against the frames; or in softer materials, after setting a frame, the sheeting is driven around it on the outside, and driven ahead so as to keep in advance of the excavation. When the sheeting shows signs of springing or bending another frame is inserted and another set of sheeting started between the last frame and the sheeting from above. With brick or concrete cylinders the bottom must rest on a timber or iron curb, consisting of a short cylinder of timber or iron framed with a cutting edge, and on top a ring of timber or iron of sufficient width to carry the masonry, and supported by brackets fastened to the sides of the curb. Though often used for foundations, such methods are more generally applicable to sinking shafts in mining operations, or in connection with driving tunnels, and constructing piers for bridges. Iron cylinders were used in founding the City Hall of Kansas City, mentioned in another page. Hollow brick, concrete and iron cylinders for piers of bridges will be described later.

ARTICLE LVI.

FOUNDATIONS FOR HIGH BUILDINGS.

76. IN the last few years the construction of high buildings in cities has rendered necessary a more careful and thorough examination into the bearing power of soils and remodelling the underground columns and supports, so as to secure safe bearing areas and at the same time so reduce their cross-sections that they may occupy as little space in the underground compartments as practicable; and perhaps more thought has been given to this subject and greater developments in this direction have been made in the city of Chicago than anywhere else. Until very recently it has been supposed that the clay was underlaid by a thick layer of quicksand or some soft material. The practice has been to guard carefully against cutting through this clay, and as the heights of the buildings have been increased the bases of the walls and column supports have been gradually spread and enlarged, so as to maintain a unit pressure not exceeding from 3000 to 3500 lbs. per square foot. Under some structures piles have been driven, as it was feared that the limit of safety had been reached for sufficient support by direct bearing. This has not been considered in some cases as entirely satisfactory or even as an improvement over the former method, unless the piles are made long enough to reach to the bed-rock, which is from 50 to 60 ft. below the surface, and are driven at the bottom of excavations 15 or more feet deep, so as to insure that all wood-work, piles, caps, and flooring, when used, should be certainly below the line of constant moisture. It is claimed that careless driving was the cause of the inefficiency of the piling. Following upon this, Gen. William Sooysmith read a paper arguing that some method of reaching the rock should be adopted, and pillars of stone with polished beds, so as to do away with mortar, should be used to bring the foundation up to or near the surface of the ground, claiming that such a pillar would be four times as strong as one of ordinary masonry. These methods were argued against as being entirely useless and unnecessarily expensive, and the claim is set forth that Chicago is underlaid by a solid bed of compact clay from the surface to or near the rock, passing into compact gravel immediately above the rock, and that the borings thus far made have been deceptive on account of the water which, though existing in small quantities in the clay, collects in the pits and leaves the impression that the underlying soil is either quicksand or mud. It seems to the writer borings conducted as explained in the second part of this volume would settle this matter definitely and satisfactorily, as the material can be brought up just as it exists from any depth, if it is silt or clay. It is admitted that proper, efficient, and systematic borings have not been made.* By these parties it is claimed that by the use of the combined steel and concrete beds a sufficient spread of base can be obtained to bear safely any height of building likely to be required. And so this matter stands, ably argued on both sides, but, as it seems to the writer, without sufficient and reliable data being determined to settle the question. It will be of interest, however, to see what has thus been accomplished. The economizing in the question of cellar spaces is well illustrated in Figs. 59 and 60. Fig. 59 shows a masonry pier resting on a concrete base; Fig. 60 shows a steel rail and concrete footing resting on an equal mass of concrete, therefore having the same ultimate bearing capacity. The masonry above the concrete is 7 ft. high (see Fig. 59); in Fig. 60 the height is only 2 ft. 6½ ins., the upper course being 15-in. eye-beams. A similar construction, using rails for the upper courses and transmitting the same weight, would be only 1 ft. 8 ins. high above the concrete. In this case the weight of the masonry base is 216,000 lbs.; the weight of the steel base, 103,000 lbs. The weight on this foundation is about 800,000 lbs., the weights of the foundations being respectively 20 and 13 per cent of the total. The saving in weight of the iron and concrete

* Since writing the above Mr. A. Gottlieb has made a large number of borings; about the average results will be found in the supplement to this volume. A full report was published in the *Engineering News*.

foundations is enough to allow an additional story. There is also a saving in time. The cost of constructing the stone foundation is a little less than the steel, but the increase of rental space more than compensates for this. The steel beams also enable the load to be distributed over a part of the area between two

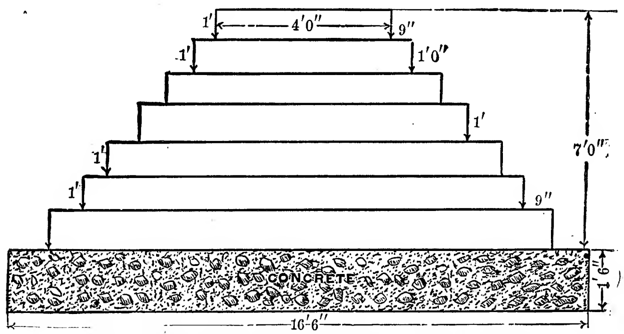


FIG. 59.—MASONRY FOUNDATIONS ON CONCRETE BASE.

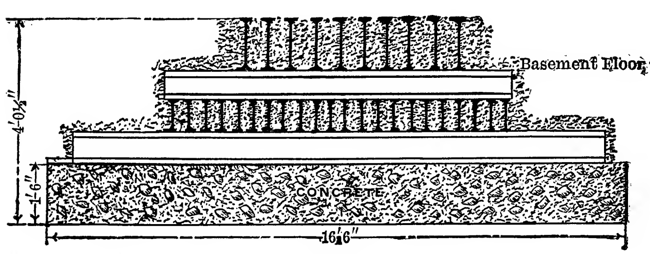


FIG. 60.—IRON RAILS AND BEAMS ON CONCRETE BASE.

columns by beams extending from the one to the other, thereby bringing into bearing a part of the foundation that could not be utilized for this purpose in masonry columns. The concrete used is of the best Portland cement and broken stone—1 part cement, 2 sand, and 4 stone. The steel rails are 75 lbs. per yard. When beams are used, the 10, 12, 15, or 20 in. beams are best. The following calculation applies to Fig. 16, the concrete bed being 17 ft. 3 in. × 22 ft. 8 ins., somewhat larger than drawing: In the eye-beams, 20,000 lbs. extreme fibre strain is allowed, and for the rails, 16,000 lbs. The top course is composed of 15-in. steel beams, 50 lbs. per foot, whose moment of resistance is 117,700 ft.-lbs.; the other courses steel rail, 75 lbs. per yard, $4\frac{3}{4}$ ins. high and an equal width of base, having a moment of resistance of 12,100 ft.-lbs. It is required to find the projecting arms of the two upper courses. Those of the two lower courses are determined from the lengths of the upper ones and the clay areas already determined.

For the two upper courses

$y =$ projecting arm;
$l =$ total load;
$a =$ width of supported area;
$M =$ total bending moment on one side of the load.

Then total length of beam $= 2y + a$,

$$\text{Total load on } y = l\frac{y}{2y + a}$$

and as the load in every course is uniformly distributed,

$$M = \frac{ly}{2y + a} \times \frac{y}{2} = \frac{ly^2}{2(2y + a)} = R. \qquad \text{(Eq. 1.)}$$

In calculating the two lower courses y becomes the known and M the unknown quantity. The load on the column is 1,166,000 lbs. As only nine beams can be put under the cast bed-plate, $M = R = 117{,}700$ lbs. $\times 9 = 1{,}059{,}300$, then $\frac{1{,}166{,}000 \text{ lbs.} \times y^2}{2(2y + 5)} = 1{,}059{,}300 \,.\, y = 5$ ft. 4 ins.; length of beam $= 2y + 5 = 15$ ft. 8 in. For the third course, $M = R = 12{,}100 \times 31 = 375{,}100$ lbs. This spaces the rails 6 in. centres. The load is $1{,}166{,}000 + 19{,}000$ (weight of top course and concrete) $= 1{,}185{,}000 \,.\, y = 2$ ft. 6 in. The area covered by the first course must be 15 ft. 11 in. $\times$ 21 ft. 4 in., giving 3 ft. $\frac{1}{2}$ in. projection for the first course and 2 ft. 10 in. for the second. Then $\frac{1{,}200{,}000 \text{ lbs.} \times 2\frac{5}{6} \times 1\frac{5}{12}}{21\frac{1}{3}} = M = 225{,}780$. Requiring nineteen rails in course second, and $\frac{1{,}220{,}000 \times 3\frac{1}{24} \times 1\frac{25}{48}}{15\frac{11}{12}} = M = 343{,}000$, or, in the first course, twenty-nine rails. Thirty were used. The allowable clay loads vary from $1\frac{1}{2}$ to 2 tons.

With this load the structure will settle from 3 to 5 ins. After carefully laying the rails and concrete the entire exposed surface is plastered over with cement mortar, so that no part of the iron is exposed. (See article in *Eng. News*, Aug. 8, 1891, by C. T. Purdy, C.E.) This form of foundations is probably the best practice for high buildings, and therefore it is given in some detail. The great advantage of this method is in only requiring a small thickness of concrete. The failure of the City Hall was due to a too thin bed of concrete and being required to act as a beam, owing to the unequal resistance of the clay. The building settled unequally, as much as 14 ins.

The following are examples of loads actually borne: A stick 12 ins. square on micaceous sand did not settle perceptibly under a load of 10 tons. And 8 tons per square foot on screw-disks at Coney Island. East River Bridge, $6\frac{3}{4}$ tons on sand. In New York approach 1600 feet masonry, $3\frac{1}{2}$ tons to $4\frac{1}{2}$ tons; no cracks. Clay under Capitol at Albany, 2 tons per square foot; 3 ft. below surface. Bridge at London, on gravel over blue clay, $5\frac{1}{2}$ tons per square foot, failed after many years. The Washington Monument when one third built caused pressure of 5 tons per square foot on a mixture of clay and sand, but settled after a number of years, $1\frac{1}{4}$ ins. out of plumb. The base on resuming work was spread by cutting channels in under the masonry, so as to reduce the pressure to 10,000 lbs. per square foot. It is estimated that this pressure is doubled on the leeward side in high winds. No evidence of further settling. Public works in India do not settle on silt and alluvium with 1 ton per square foot; with 2700 lbs., settled $\frac{1}{4}$ inch; with 2 tons, decided settlement. Fort Livingston, Mississippi, built on fine sand, 20 ft. thick, settled during building 2 to 3 ft., subsequently 1 to 2 ft. gradually. Government building at Chicago settled during thirteen years 6 to 18 ins.

Tay Bridge, Scotland, on silty sand, load 3 to $3\frac{1}{2}$ tons per square foot; weights added increased the load to 5 and to $5\frac{1}{2}$ tons per square foot, remaining from 6 to 25 days, settled about $1\frac{1}{4}$ to $1\frac{3}{4}$ ins. Exhibition buildings at Paris, gravel resting on stiff clay, 6000 lbs. per sq. ft. when the gravel was 10 ft.

thick, 4550 lbs. when 5 to 10 ft. thick; when less than 5 ft. thick, piles were driven. In one test a load 8 tons per square foot, caused a settlement of 11 ins. in 12 hours, but 6 tons were carried safely. Hudson River Tunnel, on Hoboken side, safe load on mud 5580 lbs. per square foot.

A length of 9 ft. 6 ins. in a wall settled about $\frac{1}{2}$ inch with a pressure of 62 lbs. per square inch at time of building, the wall being built rapidly. Loading walls too quickly has caused bulging. Building masonry with very thin joints on the face and thicker joints behind often causes chipping on the face, notably Philadelphia Public Building and Washington Monument (see *Engineering News*, Feb. 14, 1891).*

The Eiffel Tower: total weight of iron, 7300 tons. The total load on foundations is 565 tons, increased to 875 under maximum wind pressure. Total height of the tower, 984 ft. There are four independent foundations at the angles of a square 330 ft. on a side, and each foundation is made up of four separate inclined piers. The main foundations are on a bed of gravel 18 ft. thick, the top of the bed 23 ft. below the surface; these rested directly on a bed of concrete 7 ft. thick. For two of the piers the bed of sand and gravel, about 40 ft. below the surface, overlaid by soft deposits, was reached by the use of compressed air, the caisson being sunk 52 ft. to a good bearing soil. The bed stones under the great piers have a crushing strength of 1600 lbs. per square inch; maximum load that can come upon them is 425 lbs. per square inch. The total load on each of the two foundations on the concrete bed is 1970 tons. The concrete has the following dimensions: 32 ft. 9 ins. $\times$ 19 ft. 8 ins. (= 644.86 sq. ft.) and 6 ft. 6 ins. thick. The load on the masonry is about 3 tons per sq. ft. (see *Eng. News*, June 8, 1889).

The City Hall of Kansas City was constructed over the site of an old ravine, which was partly filled by the material from the adjacent clay bluffs, and in part by the ordinary rubbish

* This settlement of brick walls and buildings refers to settlement or shrinking of mortar in joints of masonry either causing settlement of whole structure uniformly, or unequal settlement due to difference in thickness of the same mortar joint, throwing an excess of pressure on the face, causing chipping.

dumped in from the city carts; the fill was 50 ft. deep. Holes were bored by means of a large auger 4 ft. 6 ins., worked by steam; an iron cylinder, metal thickness $\frac{3}{16}$ in., followed the auger down; when solid bottom was reached the cylinders were filled with vitrified brick well bonded; these bricks had a crushing strength of about 135 tons each.

The Chicago Auditorium Building has a frontage of 362 ft. Total area covered about 63,000 sq. ft. The building proper is 10 stories high; on one of the fronts a tower rises 240 ft., and 94 ft. above the main building. The foundation of the tower covers an area of 69 $\times$ 100 ft. The weight on the foundation-bed, 15,000 tons, = about 4350 lbs. per sq. ft. An excavation was made to the clay layer; on this bed a timber grillage, 2 ft. thick, was constructed; on this, solid concrete 5 thick was placed; and to prevent unequal settlement and distribute the weight uniformly three layers of rails, one layer 15-in. V-beams, and one layer 12-in. eye-beams were imbedded in the concrete; and as an additional precaution against the heavy, concentrated weight of the towers cracking the adjoining walls a direct load, about equal to its completed load, was placed on the tower walls, and gradually removed as the walls were carried up.

Many of the high buildings have foundations of this character. Where timber is used great care should be taken to place it well below the surface of constant moisture.

In all of the above examples, except where specially noted, the structures have stood, no serious settlement having occurred up to the time of publication. Many other similar examples could be cited, but the above shows the usual loads, methods of founding, and nature of the structure, taken from many localities, and constructed on many varieties of material.

The writer constructed the high masonry piers for a bridge over the Ohio River on this plan, bedding, 12 $\times$ 12 in. Michigan white pine in the concrete. Whether either timber or iron is preserved from rot and corrosion when entirely covered in concrete can scarcely be considered as a settled fact, though generally accepted and believed. See Fig. 17.

77. In the above cases the pressures on the foundations are in the main given without any complications or uncertainties resulting from frictional resistances on the sides of the structures, which exist in the case of pile foundations or those constructed by the open crib or pneumatic caisson. The published accounts of these are liable to be uncertain and misleading, both on account of the inaccurate distribution of the total resistances between the direct bearing resistance and the frictional resistance, which is varying within wide limits, and also from the different manner of calculating the actual load to be supported. In sinking an open crib or pneumatic caisson some engineers deduct from the total weight the actual or assumed buoyancy of the displaced water as well as that of the displaced earth, and others do not. This results in a wide difference in the resultant load to be supported by friction and direct bearing. Some uniform method is necessary for an intelligent comparison or subsequent use in other structures. In the case of the foundations of the Cairo Bridge, already described, the calculation is made as follows:

Channel Piers.	Tons.
331,000 ft. B. M. timber at 50 lbs. per cu. ft.	689.6
Iron	68.5
Concrete, 77,345 cu. ft. at 145 lbs. per cu. ft.	5,607.5
Masonry, 102,508 " " " 150 " " " "	7,688.1
Superstructure	1,027.0
Moving load	785.2
	15,865.9
Deduct for displacement of 78,000 cu. ft. sand, and 22,756 cu. ft. water, and frictional resistance at 400 lbs. per square foot	9,574.5
	6,291.4

Assumed friction resistance 400 lbs. per square foot.
Fatigue weight 6,291.4 tons = 3.15 tons per square foot.*

* In this calculation concrete at 145 lbs. per cu. ft. seems high, 135 lbs. is doubtless a good average. Masonry at 150 lbs. per cu. ft. is low for first-class masonry, average about 155 to 160 lbs. Timber at $4\frac{1}{6}$ lbs. per ft. B. M. = 50 lbs. per cu. ft. = 2.09 tons per 1000 ft. B. M. is a fair average. Taking sand at 130 lbs., when wet, water at $62\frac{1}{2}$ lbs. per cu. ft., then $(78,000 \times 130 + 22,756 \times 62\frac{1}{2}) = 5781$ tons. $\therefore 9574.5 - 5781 = 3793.5$ tons for total frictional resistance.

Also for load on concrete base, pier 12 (supposed to be a pier on land):

	Tons.
12,072 cu. ft. masonry	905.4
Superstructure	234.6
Moving load	379.5
	1,519.5

Or 4.03 tons per square foot.
The fatigue weight = total weight less displacement and friction.

From report of Mr. Geo. S. Morrison on the construction of the Bismarck Bridge (which contains much detailed and valuable information) the following is extracted: Pier No. 4 on land, excavation in sand to a depth of about 20 ft., and piles driven in the bottom.

	Lbs.
28,000 ft. B. M. timber in curb at 4 lbs	112,000
15,000 " " " " grillage at 5 lbs	75,000
264 cu. yds. concrete at 3,510 lbs. (130 lbs. per cu. ft...	926,640
1093.3 cu. yds. masonry at 4,330 lbs. (160 lbs. per cu. ft.	4,733,989
257 ft. superstructure at 5,000 lbs	1,285,000
	7,132,629
Deduct for immersion 11,390 cu. ft. at 62½ lbs	711,875
Net weight	6,420,754
161 piles, average load per pile	39,880

Also pier No. 2, pneumatic caisson, sunk through water, sand, and into a hard black clay. The total weight, as calculated in the same manner above, = 17,269,000, and deducting for immersion 4,510,000; net weight = 12,759,000 lbs.; area of base, 1924 sq. ft.; average pressure per square foot, 6631 lbs., or 46 lbs. per square inch.

In the first case a steam pile-hammer was used in driving the piles: depth driven in sand varied from 23 to 34 feet. The penetration in the last 10 blows varied from 0 to 0.2 of a foot, or an average for the greater penetration of 0.02 ft.

Similarly in the Plattsmouth Bridge one of the piers on a pile foundation has the following record: Total weight, 2,114,750 (apparently no deduction for immersion); number of piles, 78; average weight on pile, 27,112 lbs. The piling record is especially interesting. Weight of hammers, 3100 and 3900 lbs.; average fall at last blow, 28 ft.; average penetration at last blow about $1\frac{5}{8}$ ins., some few as much as 2 to 3 ins.; average depth in the sand, 27 ft. Out of the 78 piles, 8 broke off under the blows; 15 piles broke, mashed, and split or otherwise injured; that is, about 10 per cent were broken off and about 20 per cent were visibly injured. The number of blows ranged from 100 to 142 per pile. It can hardly be doubted that there were many piles more or less seriously crippled below the surface and out of sight.

The other piers being founded on rock, the pressure per square foot is unimportant. They ranged, however, from 4090 lbs. to 6393 lbs. per square foot. No allowance for nor notice of frictional resistance seems to have been allowed for in these reports. Although Mr. Morrison deducts for the immersion of the structure, he says that this is only done to get the relative pressure, that is, the increased pressure on the foundation over that on the surrounding surface, "which is the real measure of the labor of the foundation." The actual pressure is the whole weight of the structure "with the addition of the atmospheric pressure." The relative pressure might be useful in estimating the bearing resistance at one depth in a certain material, knowing the bearing resistance at any other depth, provided the law of variation, increase, or decrease was proportional to the displacement; but this could only be true in a perfect fluid. If this is true no allowance for side friction should ever be made. Some engineers do not consider friction at all, owing to the fact that it may be in part or entirely destroyed by scour. That it does, however, form a very large proportion of the actual bearing resistance of many structures, and often is the sole reliance cannot be denied. Other examples of pressure on foundation-beds have been given in the preceding pages.

78. The determination of the frictional resistance on the surfaces of piles, cribs and caissons has not been considered as carefully as the importance of the subject demands. Favorable conditions do not always present themselves, and exact conditions are not always known, friction during continuous motion being different from that developed in starting from a condition of rest, and again varying with the period of rest; and often high average surface resistance may result from great local resistance at only a few points. The following are a few examples of the estimated frictional resistances by different engineers, from data and other considerations satisfactory to them. But to be convinced of the value and importance of friction, endeavor to pull piles and find the power necessary, after due allowance for weight of pile, the effect of suction, and want of rigidity in the fulcrum, and this too notwithstanding that smaller cross-sections are being exposed, and only after a considerable amount of lift will the pile be raised readily and rapidly. On measuring friction of motion by sinking piles with weights, the loss of frictional resistance is not so apparent, as larger surfaces are being pressed where the smaller were; and this largely explains the fact that pile foundations generally settle only a short distance before a new condition of equilibrium is brought about. And as is often stated that it is only the initial resistance to settling that is worth considering, it is evident that the above-mentioned fact has been overlooked. In sinking a pneumatic caisson or crib, as smaller surfaces are being continually presented in the place of larger ones, the initial resistance is soon dissipated entirely, and the tendency is to continue going when once started. This is entirely in keeping with the experience in the Hawkesbury caissons, where those without any bottom spread were the more readily handled, and gave less trouble. In measuring the resistance to sinking caissons, it is the exception, rather than the rule, that the compressed air is entirely removed from the caisson. This would reduce the resistance by the air-pressure per square inch or square foot. The escape of the compressed air under the caisson may materially reduce the outside friction. In many cases, however, the air finds a passage of escape at considerable distances from the caisson, as evidenced by the bubbling of the water at the surface. It is clear, then, that estimates made on the frictional resistance cannot, under the ordinary conditions and manner in which they are calculated, be regarded as by any means accurate. But more careful and extended observations should be made when opportunities occur. The circumstances under which the following estimates were made are not known to the writer, except those made by himself. It is, however, all that he has, and is given simply as stated in *Engineering News* and other works. Mr. Collinwood in the *News* of Feb. 21, 1891, states that in sinking brick wells 5 to 18 ft. diameter to a depth of 50 ft. or more, a load of 300 tons was required; in the alluvium in India the "skin friction" was from 500 to 1500 lbs. per square foot. At the Dufferin Bridge it was 1000 lbs. for wells 12½ ft. in diameter. On the caissons of iron of the St. Charles Bridge, sunk through 20 ft. of bowlders, the friction was 466 lbs. per square foot. The caissons of wood, East River Bridge, gave about 900 lbs. in bowlders, clay, and sand, and in clear sand from 400 to 600 lbs. per square foot. And in case of iron caisson for a lighthouse, 200 lbs. per square foot.

In the case of caisson No. 2, Susquehanna River, sunk under the writer's supervision: Indicated air-pressure of 33 lbs. This was reduced to 26 lbs. in order to sink the caisson under a total weight of 9,356,760 lbs.; deducting the buoyant effect of the air = 6,143,904 lbs., the net weight to overcome resistance = 3,212,856 lbs., giving 331½ lbs. per square foot of exposed surface in sand. A subsequent test gave 380 lbs. per square foot. At this depth the caisson had entered into a layer of bowlders which continued as far as the caisson was sunk, 68′ 4″ below low water, about 55 ft. through a rather clean sand and underlying bowlders. The material was entirely removed from under the caisson, and it was held by friction alone. The only source of error in the last case (380 lbs.) was inaccuracy in the pressure-gauge. The indicated pressure was reduced to that due to the depth; this was done as in ordinary sand and gravel it is difficult to maintain a pressure greater than that due to the depth, on account of leakage. Had a similar reduction (5½ lbs., gauge 36 lbs.; depth below water about 60 ft.; estimated actual pressure $30\frac{1}{2}$ lbs.) been made in the first case, the two records ($331\frac{1}{2}$ and 380 lbs.) would have been nearer together, as they should have been. The weights were the same, and the depths being $44\frac{1}{2}$ and 47 ft., respectively. In sinking caisson No. 3 the two records show at a depth of about 40 ft. below the bed of the river in good grained sand, weight 8,465,871 lbs.; buoyant effect of air, 6,032,989 lbs., reduced weight to 2,432,882 lbs.; area of surface below the bed of the river, 8533 square feet; hence friction resistance per square foot 285 lbs. When $46\frac{1}{4}$ ft. down record shows 379 lbs., having entered the bowlders. The caisson was sunk about 4 ft., and building commenced on the bowlders. Depth below bed of river $50' 8\frac{3}{4}''$. Total below water surface $70' 8\frac{3}{4}''$.

At caisson No. 4, depth of water, 28 ft. at low-water; depth of solid material to highest point of rock, $31' 10\frac{1}{2}''$; to owest, $37' 3\frac{1}{4}''$; which was a compact silt, air-tight and water-tight, but easily forming mud or slush when mixed with water. When the caisson first rested on the bottom (as it had a false bottom for launching; none of the other caissons had one), it rested easily, but sunk 3.3 ft. while cutting out false bottom. The ordinary weighting with timber and concrete was sufficient to sink the caisson into the bed about 14 ft. by only reducing the pressure about 3 lbs.; and when the cutting edge was 57.5 ft. below water, or 29.5 ft. in the silt, the caisson settled under a reduction of only 1 lb. in the air-pressure, showing a nice adjustment between weight and resistances. At this depth, total weight 10,958,448 lbs.; upward pressure of air, 9,014,907 lbs.; resistance to sinking, 1,943,541, giving 308 lbs. per square foot. In settling 1.3 ft. farther, one edge of the caisson rested on rock; and as the caisson was out of level about 15 inches, the rock was blasted off along this end; then reducing the pressure to level the caisson, the frictional resistance was apparently $489\frac{1}{2}$ lbs.; but the caisson was blocked against the soft material on the lower side of the caisson to prevent settling there; this record is then too high. The caisson was stopped at $65' 3\frac{1}{4}''$. below low-water; the excavation carried on, without difficulty, below the cutting edge to rock on all sides. Greatest depth to rock, 69′ 10½″.

In caisson No. 8, depth of water 29 ft.; of soft silt at highest point of rock, 47 ft.; at lowest, 59.34 ft. Caisson was only sunk 3 or 4 ft. into rock at highest point. Excavation carried on below cutting edges, exposing rock over whole area. Least depth below low-water, 76 ft.; greatest, 88.34 ft. Owing to the large size of this caisson, the softness of the silt, and apprehension of trouble, the material was never removed entirely from under the caisson ; in fact, it was sunk resting on blocking to a great extent, so it was impossible to estimate the frictional resistance with any degree of accuracy. The caisson sunk almost continuously, although concreting was stopped when within 29 ft. of the rock, the only extra weight being the necessary timder to keep the top above water as the caisson settled. The frictional resistance could not have exceeded 200 lbs. to the square foot, owing to the slimy nature of the material passed through. The depth below high tide would be about 92 ft. The other depths for caissons 2, 3, and 4 should also be increased by the same amount, as this latter was the depth for which air-pressure had to be provided every day. In high winds the tides would be either very high or very low. The following observations were made in sinking the caissons for the Cairo Bridge :

Penetration in sand, 86.42 ft.; below water surface, 90.27 ft.; weight of caisson, 887 tons : crib, 3163 tons ; masonry, 2800 tons ; weight of sand and water, 1806 tons ; total, 8656 tons.

Indicated air-pressure before sinking			42.75 lbs.
Calculated " " " "			39.117 "
Indicated " " " " when lowered			36.00 "
Air-pressure	42.75,	39.117,	36.00 "
" " reaction due to	4,802,	4,394,	4,044 tons
Net weight	3,854,	4,262,	4,612 "
Exposed surface of caisson	12,910,	12,910,	12,910 sq. ft.
Frictional resistance	597,	660	715 lbs. per sq. ft.

However, in finding the "fatigue" weight (see par. 77), or pressure on the foundation bed = total weight — deductions for displacement of sand and water and frictional resistance on exposed surface of caisson, they only allowed 400 lbs. friction per square foot. Assuming wet sand to weigh 140 lbs. per cubic foot, and water $62\frac{1}{2}$ lbs., we find the weight of the displaced material $= 78{,}000 \times 140$ lbs. $+ 22{,}756 \times 62\frac{1}{2}$ $= 6171$ tons, and side friction $= 9574.5 - 6171$ tons $= 3403.5$ tons, which would give, at 400 lbs., 17,017.5 square feet of surface in that case. The data is not given, but this shows the method of calculation adopted.

79. There seems to be very few records in regard to the frictional resistance on the surface of piles. In what is in fact a liquid mud the resistance to settling has been fully shown to be not less than 130 lbs. per square foot, and in compact silts and clays 200 to 250 lbs. would not be excessive, though not based upon actual experiment. As in sinking caissons, which would certainly give minimum values for the reason stated, 300 to 400 lbs. per square foot is a fair resistance, and as this increases as piles settle, we can safely allow from 300 to 500 lbs. for piles in sand and gravel.

In a letter from the city engineer of New Orleans he states that the soil is alluvial—a sandy clay saturated with water at a depth of three or four feet. Allowing 1000 to 1500 lbs. per square foot, if the spread is not more than ten bricks, the brick wall is simply started on the bottom of the trench. If piles are required, they are driven 4 ft. centres and capped with a four-inch floor, upon which the brick-work is started. Piles from 25 to 40 ft. long will carry from 15 to 25 tons, with a factor-of-safety of 6 to 8. Taking, then, a pile of 25 ft. into the ground, and assuming average diameter 12 ins., having then 3 square feet to the linear foot, or 75 square feet of surface, the direct bearing being taken at 1500 lbs., the safe load on the pile will be $15 \times 2000 = 30{,}000$ lbs., or to be carried by friction, $30{,}000 - 1500 = 28{,}500$ lbs., and frictional resistance per square foot will be 380 lbs., and for the 40-ft. piles 405 lbs. per square foot. This is in excess of the suggested allowance by $405 - 250 = 155$ lbs., for such material.

The enormous grain elevators in Chicago rest upon pile foundations. Mr. Adler states that the unequal and constantly shifting loads are a severer test upon the foundations than a static load of a 20-story building. Taking the load on the steel and concrete piers already illustrated, the concrete bed is 17 ft. 3 in. $\times$ 22 ft. 8 ins.; with piles 2.8 ft. centres we could get six rows of nine piles each = 54 piles; with 3 sq. ft. per linear foot a 50-ft. pile would expose 150 sq. ft. of surface at a frictional resistance of only 100 lbs.; each pile would carry 15,000 lbs., and the 54 piles would carry 810,000 lbs. On the concrete and steel foundation the load is 800,000 lbs.; at $2\frac{1}{2}$ ft. centres about 70 piles could be driven in the same space. With the low limit of 77 lbs. per square foot a 50-ft. pile would carry 11,550 lbs., and the 70 piles 808,500 lbs. With the knowledge that we have there can be but little doubt, if any, that a pile foundation will carry any loads yet put upon the soil underlying the city of Chicago, if properly arranged and driven.

80. The reader will have learned, if he has even casually glanced over this volume, that engineers and architects are far from agreement as to the mode of determining the bearing power of any of the materials upon which we have to build; we are far from agreement as to the safe loads that can be put upon them or the proper manner of distributing the loads. In such cases arguments are useless; high-sounding or ingenious formulæ are of but little value. Theories will not solve the problem.

What we need is systematic, honest, extensive experiments and tests, and with these, honest, impassionate interchange of ideas and deductions, without petty jealousies or fault-finding. With rigid but kindly criticism of designs and of methods of construction, we might hope to advance our knowledge, improve our practice, and give the public safe, substantial, and satisfactory results, at the least cost and in the least time.

In writing this volume the writer has endeavored to avoid putting forward pet plans or theories. If prominence has been given to designs, it is only because he believed them as good as those of others, and was more familiar with their details; and with simple alterations in some of the details they are typical of all such structures. He has commented on the designs of others, expects such criticisms of his own, as only in this way can we hope to arrive at the truth.

ARTICLE LVII.

CONCLUSIONS.

81. The increasing demand for high buildings, owing to the contracted areas upon which buildings must be erected and the enormous cost of the same, has naturally led to much discussion, many eminent engineers and architects claiming that the limiting height and consequent weight per square foot of bearing surface has been reached. An additional spread of base beyond that now attained is impossible on account of the contracted areas and also on account of the rights of abutting property-owners. And that owing to the already great unit pressures on the usual foundation-beds of sand, gravel, or clay, which, although they now apparently safely carry their loads without any but a very small and allowable settlement, yet such structures may be regarded as in a precarious condition if subsequent operations of abutting property-owners, such as tearing down existing buildings and excavating to greater depths, either for increased cellar room or to secure better and stronger foundations, should remove lateral support from the foundation-bed of an adjacent building. The result would be a flow or bulging of the material, and thereby causing serious cracks or other permanent injury to the building. Especially is this a living danger if the material is a water-bearing sand or silt.

82. The substitution of piles as a means of spreading the base or acquiring increased bearing resistance is advocated by many equally eminent engineers and architects, backed by many well-established precedents. On the contrary, however, examples of structures on pile foundations are not wanting to show that, for some not explained or inexplicable reason, after

carrying a load for years with perfect safety, such foundations have ultimately failed, causing either a partial or complete wreck of the structure.

Whether or not pile foundations are any better, so far as affording direct support is concerned, may be a matter of reasonable dispute and difference of opinion, as piles may be badly injured in driving, their upper portions may not be driven below surfaces of constant moisture, or, if so driven, the superincumbent weight may lower the level of this water-surface; or subsequent systems of drainage, or even natural subsidence of the surface, may occur, exposing the piles to conditions of alternate wetness and dryness, resulting in the piles rotting and consequent failure of the structure. A case of this kind occurred under the tower of a market-house in the city of Richmond. The building was badly cracked. Col. W. E. Cutshaw, City Engineer, on excavating below the structure found that it had been originally built on piles, which had rotted to a considerable extent and depth below the surface. After forcing the walls back, closing thereby the cracks to a great extent, and supporting them in this position with jacks and props, he excavated under the walls and filled the trench with a carefully made cement concrete, rammed in layers, which were allowed to partially set as the work progressed, so that when the concrete was rammed under and against the old walls the entire mass had a good set. Subsequently the props were removed, throwing the entire weight on the concrete. No further trouble has occurred.

83. Although this and similar cases show the uncertainty, under some conditions, of pile foundations, they are used to a great extent with entire satisfaction, and will certainly to a very great extent do away with the danger of the material flowing or bulging from under a structure by removing the weights from or excavating in adjoining lots.

We have, then, open to the builder for selection:

1st. Simply building the foundation walls or pillars on the natural bed, spreading the base with projecting courses of masonry.

2d. Obtaining the necessary spread with a timber platform or grillage.

3d. Driving piles, either to some hard or firm material or to rock.

These two methods may cause settlement by rotting of the timber.

4th. Building the walls upon beds of concrete of sufficient area, either alone or strengthened by iron or timber beams built in the concrete.

5th. Sinking cylinders of iron or caissons of timber or iron of such dimensions as to support either a single column or a series of columns or walls, these caissons being sunk either to rock or to such a depth and material as will preclude the possibility of failure occurring from any of the above-mentioned causes.

84. Two notable examples of this last method are found in the City Hall of Kansas City, in which case cylinders were sunk to rock, as explained, and the new pump-house of Louisville Water-works, which is erected on a large timber caisson sunk by the pneumatic process, a single caisson of sufficient dimensions for the entire building to rest on being used.

85. A somewhat new departure in this direction will be found in case of a large building, the Manhattan Life Building, shortly to be erected in New York. Sooysmith & Co. are the contractors, and they have kindly sent me in advance of the commencement of the work, or even the construction of the caissons, the following facts and data. My excuse, if any is needed, for inserting any account of a foundation not even commenced in a work on foundations is the perfect assurance that the work will be carried to a satisfactory completion, whatever may be the difficulties or costs involved, by the contractors.

Depth to bed-rock is 50 ft. below the level of Broadway and 25 ft. below the cellar floor, which is even with the natural water-level. The caissons will be sunk by the pneumatic process through sand and quicksand. There will be fifteen caissons built of boiler-plate steel. Four (4) of the caissons will have circular cross-sections, varying from 10 to 15 ft. in diameter. The remaining eleven (11) caissons will be of rectangular cross-section, ranging in size from 13 to 26 ft. square, some of them being 10 ft. × 26 ft. in dimensions. The roof will be strengthened by 15-in. eye-beams. The piers will consist of hard-burnt bricks laid in Portland cement, capped with several courses of granite, stepped off to a proper size to receive the bed-plates for the iron columns, some of which will support as much as 1500 tons.

The entire number of caissons will be put in place at once and the brick-work commenced on them. Several will be sunk at the same time. It is the intention of the contractors to sink these caissons with a weight sufficient to overcome both frictional resistance and the buoyant effect of the compressed air, and not to sink, as is ordinarily done, by reducing the air-pressure. The object of this is to prevent the large inflow of sand and gravel which usually takes place when the air-pressure is reduced.

It is highly probable that this can be accomplished, as the depth to be sunk is not very great, and both the total frictional resistance and upward pressure of air will be relatively small.

At very great depths in sand and gravel it becomes often difficult to sink the caisson, even with greatly reduced air-pressure, and in many sands and gravels more or less inflow of the material takes place even against the air-pressure, especially if the excavation is carried at all below the cutting edge. At least this is the writer's experience. The ability and experience of the contractors will, however, enable them to contend successfully with any difficulties likely to arise.

The entire question, heretofore, of applying this method to ordinary buildings has been one of actual and relative rapidity and cost in the construction. If these difficulties can be removed, there can be no question of the advisability of adopting this method for all important buildings, even if the necessity, from a practical point of view, does not require it—certainly when rock is at no very great depth below the surface, as the feeling of perfect safety as well as the demand for it will justify the increased cost, provided it is not too large a percentage of the entire cost of the structure. And as buildings become more costly the smaller will be the percentage of cost for the foundation, and the greater is the reason, purely from a selfish or economic point of view, to make the foundations absolutely safe and secure, to say nothing of the greater danger to the comfort as well as to the life of the occupants, which imposes a far larger responsibility on the builders.

86. In this connection the relative advantages, both as regards rapidity and cost of construction, of sinking wells for foundations, which is done successfully in India to a great extent, and to which allusion has already been made, may be discussed. An interesting construction of this kind has been only recently completed, an account of which will be found in the *Engineering News* of January 12, 1893. These foundations were for the piers of a bridge on the Madras Railway, India. The method, however, is equally applicable to the foundations of high buildings where it is desired to reach bed-rock. The spans for this bridge were about 140 ft. long. The masonry for the abutments and piers above a certain level was limestone, resting on foundations obtained by well-sinking.

The plan of the abutment was of the usual form, with face wall and splaying wings. Seven masonry wells resting on curbs were sunk to rock 57 ft. below the bed of the river—three wells under the face wall and two under each wing. The external diameter of the well was 12 ft. where it rested on the curb, and for a height of 20 ft. above, where it was reduced to 11 ft. diameter. The walls of the well were built of limestone masonry laid in mortar. In one of the wells, sunk 40 ft. through clay, one side rested on masonry of an adjacent well, and the other side on a curb of another adjacent well, but the excavation was carried down between the wells to the bed-rock, and the space refilled with concrete. The entire enclosed space in all of the wells was filled with Portland cement concrete. The thickness of the walls of the wells is not given, but assuming from $2\frac{1}{2}$ to 3 ft., the diameter of the enclosed hollow space would be from 7 to 8 ft., and about 57 ft. long. Arches were then sprung from cylinder to cylinder, and the solid masonry built on the arches. The piers were constructed by sinking two cast-iron cylinders, 12 ft. in diameter, for each pier placed 18 ft. centre to centre. These cylinders were from 40 to 47 ft. high, reaching from bed-rock 55 to 62 ft. below the bed of the river, to a point 15 ft. below the bed. At this point the diameters were reduced by a conical taper 7 ft. 6 ins. high to 9 ft., and carried to full height at this diameter. These were finished on top with a sliding cap and connected by wrought-iron massive bracing-boxes, bolted and filled with concrete. To facilitate the sinking of the cylinders at a point 12 ft. above the cutting edge brackets 2 ft. 9 ins. wide were fastened to the cylinders, on which a masonry wall or lining was built up to a point at the middle of the length of the conical taper. This really converted the cylinder proper into a masonry well with iron casing, reducing the external frictional resistance. The hollow spaces in all cylinders were then filled with concrete. Above the top of these wells or cylinders solid masonry was built to the top of the pier, below the bed of the river in cement mortar, and above in "surki" mortar. The total depth sunk, of cylinders and wells, was 2364 ft. The cost for the cylinders was $14,170, and for the wells $11,630, including charges for bedding on the rock performed by divers. Total cost, $306,402, or $146 per linear foot of bridge, which was 2100 ft. long. The iron in the cylinders cost $85,142, and in the girders $110,000, or total cost of iron $196,410 (as given in report); leaving for the masonry lining, concrete filling, and sinking of the wells and cylinders $109,992, or per linear foot of cylinder $46.50. This last calculation is made by the writer, as he understands the data given above.

87. Also for the end piers of the Kentucky and Indiana cantilever bridge across the Ohio River at Louisville, brick-lined cylinders of iron were sunk and subsequently filled with concrete, constituting a similar construction to the above. The pier on the Indiana side, carrying one end of a 240-ft. through-span, was composed of two plate-iron cylinders, metal thickness $\frac{3}{8}$ in., in sections, riveted together as the work progressed.

The brick lining rested on a shoe or cutting edge at the bottom about 1 ft. high; the thickness of the brick wall was 18 ins. for a height of about 6 ft.; the thickness was then reduced to 13 ins., and continued at that thickness to the top. The cylinders were sunk simply by excavating the material on the inside, and building the cylinder and brick-work at the same time. After reaching the proper depth, the hollow inclosed space was filled with concrete. The interior diameters of the cylinders at the bottom were 10 ft.; exterior diameter, 13 ft. 0¾ in.; at top interior, 7 ft. 6 in.; exterior diameter, 9 ft. 8¾ in. The excavation was carried down in the rock 5 ft. below the cutting edge. The following is the cost of this pier:

2 cylinders ⅜-in. iron, 105,662 lbs. at 5.72 cts.		$6,043 87
504.0 cu. yds. earth excavation inside of cylinder at 75 cts.		378 00
144.2 " rock " " "	$1.50	216 30
170.28 " brick-work in lining at $11		1,873 08
299.75 " concrete filling at $6		1,798 50
9.72 " coping at $20		194 40
6.5 " cut backing at $6.60		42 90
Sundries		78 59
420.0 cu. yds. earth filling around cylinders at 25 cts.		105 00
Total for two cylinders		$10,630 64

And for each cylinder $5315.32; the total length being 75.9 ft., or per linear foot of caisson $70.03. The engineer in charge, Capt. C. A. Brady, who has kindly furnished me with the above and the following data, informed me that the rock excavation was entirely unnecessary, as it was a hard, compact slate, which would reduce the cost per foot of length to $57.69.

The two cylinders for the pier on the Kentucky side were larger, carrying the projecting arm of the cantilever span 260 ft. long, as they also acted for an anchorage pier. Bottom diameter 15 ft. 5½ in. exterior, and 11 ft. at top, lined similarly to the above-described cylinders; total length 112.2 ft.; weight of iron in two cylinders 153,081 pounds. The unit prices same as above; quantities considerably increased. Total cost $18,498.36 for the two cylinders, for each $9249.18, and cost per linear foot $83.32. These cylinders were sunk through

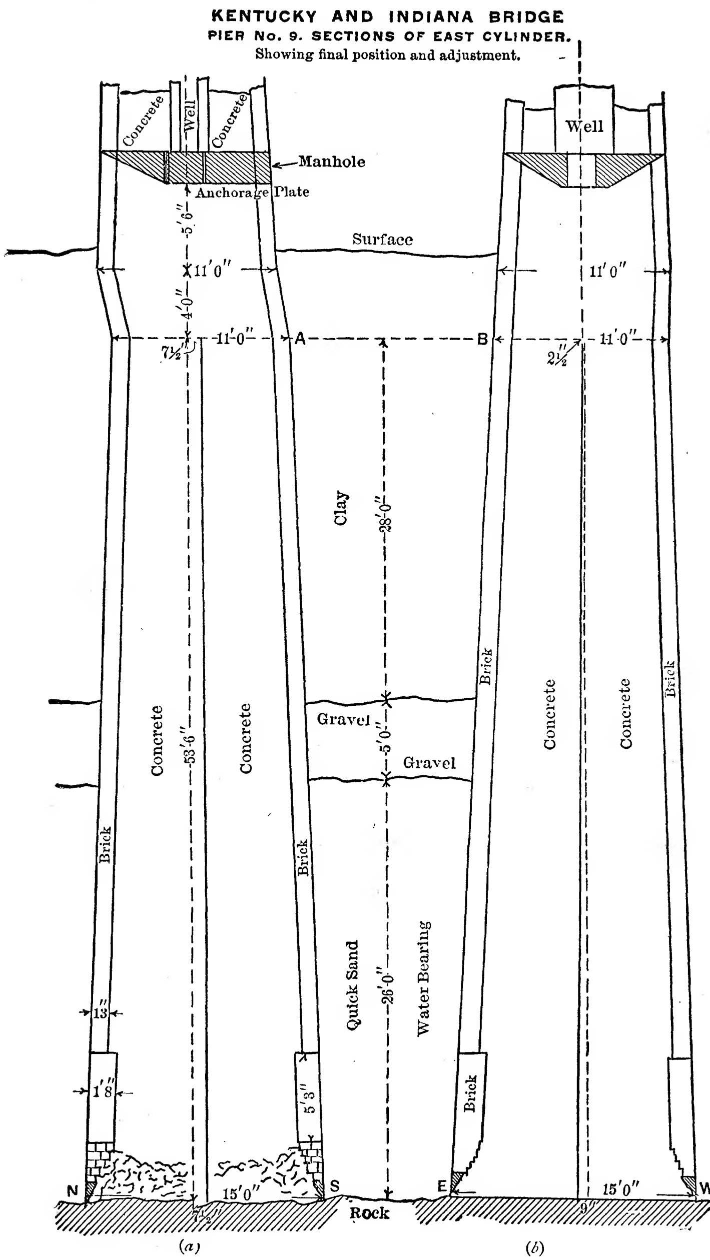


FIG. 61.—ANCHORAGE PIER FOR CANTILEVER. CYLINDERS SETTLED OUT OF LINE; ADJUSTED TO PROPER POSITION WITH INCLINED PLATES.

28 ft. clay, 5 ft. of gravel, and 26 ft. of quicksand, or a total depth below the surface of 63 ft. Owing to caving in of the quicksand, letting the material down from above, the cylinder was thrown out of position; much additional labor and cost was required in removing material, pumping and straightening cylinders, adding largely to the cost of the foundation above

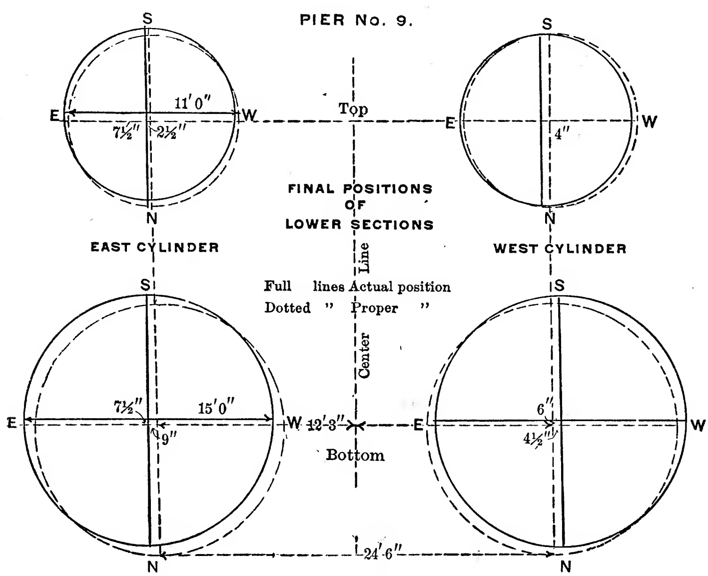


FIG. 62.—KENTUCKY AND INDIANA BRIDGE PLAN OF CYLINDERS.

an ordinary case. The cylinders were not finally straightened at all. To bring the top into position it was necessary to rivet a slightly inclined elbow, as shown is Figs. 61 and 62. Details of the anchorage connections are shown in Fig. 62, near the top. These last cylinders are considerably larger than those used in India, but the difference in cost is also very great. Such costs may be almost prohibitive for ordinary foundations for houses.

SUPPLEMENT.

THE HAWARDEN BRIDGE.

88. THE Hawarden Bridge over the river Dee, in England, consists of two fixed spans 120 ft. each and of one draw-span 287 ft. end to end. The clear opening on one side of the pivot pier is 140 ft. and on the other 87 ft. It is a double-track bridge, with a (4) four-foot-wide footway on one side. The piers are brick and concrete lined cylinders. The wrought-iron cylinder for the pivot pier was built of ½-in. plates riveted together, having a special rolled section of iron, forming the actual cutting edge, inserted between them. These plates rise vertically for a height of 9 ft. outside; but internally they slope upward toward the centre of the cylinder, until at a height of 9 ft. above the bottom edge the internal diameter is 30 ft., the diameter of the bottom or cutting edge being 43 ft., the reduction being 6½ ft. all around, forming a V-shaped curb or section 9 ft. high and 6½ ft. wide at top. This section was filled with cement concrete made 5 to 1 after being sunk on the bed of the river. Upon this bottom section two concentric cylinders made of iron plate were built to a height of 15 ft., making a total of 24 ft. in height above the cutting edge; this was sufficient to reach from the bed of the river to above high-tide line. The cylinder was then sunk in its proper place, about 200 tons of concrete being placed in the annular space between the two plate-iron cylinders. The width of this annular space was reduced to about 5 ft. above the V-shoe or section, and in it, resting on the concrete, a cylindrical brick wall 5 ft. thick was built, the internal diameter being 30 ft. and the external diameter 40 ft. This brick wall was carried well up above the water surface. Dredging was then commenced inside the cylinder, using the clam-shell dredge. As the material was removed and the brick walls were built up above the iron casing, the cylinder gradually sank. Except that the iron plating was only carried to the height of 24 ft. above the cutting edge,—the hollow brick cylinder having no sheathing above that point outside nor inside, and that the brick walls were 5 ft. in thickness and diameter 40 ft. out to out of wall,—the general construction was the same as described for the cylinders of the K. and I. bridge described in paragraph 87, Part III.

When the excavating and sinking commenced the weight on the cutting edge was about 6 tons per linear foot. The cylinder was sunk to a depth of 48 ft. below the bed of the river. With a weight of about 2300 tons the caisson could be controlled easily by careful and skilful handling of the clam-shell dredge, aided by pumping water under the cutting edge. The cylinder was sunk mainly through sand, but if it rested at any point on bowlders or lumps of hard clay or other material, it was found easy to remove these by pumping water under them. This latter process was also found greatly to reduce the frictional resistance on the surface of the iron or brick work. This same effect was referred to in discussing pneumatic caissons as due to the escape of the compressed air under the caissons and rising along its outer surface. After reaching the proper depth below the bed of the river, concrete was lowered in "pigeon-trap" boxes through the water inside of the brick cylinder and deposited at the bottom; this was continued until its depth was about 18 ft. The concrete was made with strong cement, in the proportions of 5 to 1. When this mass of concrete had set, it was found practicable to pump the water out of the cylinder, and the rest of the concreting was deposited in the dry. This concrete was mixed 6 to 1, and filled the cylinder to a height of about 65 ft. above the bottom; then a floor of brick-work in cement was laid over the whole surface, on top of which was placed a large granite block, 9 ft.

square and 3 ft. thick, for the central pivot-bearing, and also the masonry for the circular track. This is probably the largest cylinder ever sunk under the brick-well system so common in India. The cement mortar used in the brick-work seems to have been mixed in the proportions of $1\frac{1}{2}$ to 1 for the lower portion and 3 to 1 for the upper portion.

For the other piers two cylinders were used, each cylinder being about 12 or 14 ft. outside diameter and 6 or 8 ft. inside diameter, allowing brick walls of about 3 ft. thick. These were sunk to the proper depth and filled with concrete, as in the pivot piers. Three cylinders were used in one of the piers. Brick arches were then built, connecting the cylinders at the tops.

The necessary piles for fenders and other purposes were sunk by aid of water jets. A 2-in. pipe was temporarily spiked to the side of the pile, which was then placed in position; the pipe was then connected to the steam-pump by a flexible hose. The water forced through it discharging at the foot of the pile, caused it to sink with any desired degree of rapidity. During the operation of sinking the pile was easily moved or turned into any position. When the pile had reached the proper depth the pump was stopped, pipe and hose removed, and then fastened to another pile. By this method piles were sunk in sand to the depth of 20 to 25 ft. in two minutes, "without the delay, uncertainty, or damage which so frequently accompany the ordinary system of pile-driving. Sometimes a nodule of clay or erratic bowlder of the glacial drift was encountered by the pile, but by sinking another pipe down to the under side of the stone or nodule, and pumping, the obstruction sinks away in advance of the pile, which rapidly follows. Within a half of an hour of the pumping being stopped the sand settles around the pile, and no amount of ordinary pile-driving will stir it a fraction of an inch."

The total cost of this bridge was $355,000,—equivalent to $545 per linear foot. No division of this amount between the substructure and superstructure is made in the description from which the above is taken.

89. FOUNDATIONS AND FLOORS FOR THE BUILDINGS OF THE WORLD'S COLUMBIAN EXPOSITION.

On the subject of foundations for the buildings of the World's Fair at Chicago, Mr. A. Gottlieb has made an interesting report, which was published in the *Engineering News*, from which the following facts are taken.

Chicago Subsoil.—Commencing at the surface and proceeding downward, the following lay and thickness of strata are recorded in some of the many soundings made, among which there were considerable variations: Upper surface black soil, then sand 5 to 8 ft.; quicksand 4 to 10 ft.; soft clay 6 to 10 ft.; soft blue clay 6 to 10 ft.; blue clay; hard blue clay; hard-pan. Average depth to hard-pan 26 to 36 ft. below surface.

The sand, when loaded with $2\frac{1}{4}$ tons per square foot, settled $\frac{3}{8}$ in., and very uniformly, and after this there was no further appreciable settlement. On sand filling over mud holes a load of $1\frac{1}{2}$ tons per square foot settled from 1 to 3 ft., and kept sinking with the continuance of the load on the platform. In such places piles were driven. Whereas on the regular bed of sand a load of about $1\frac{1}{4}$ tons per square foot was allowed. The platforms supporting the columns of the structures were constructed of several layers of plank and solid timber scantlings, having sufficient top dimensions for the bottoms of the columns to rest upon easily, and spreading outwards and downwards, so that in all cases the area of the bottom of the platforms should be great enough to limit the pressure to $1\frac{1}{4}$ tons per square foot of surface. He also gave some interesting experiments on the resistance of timber columns under compression, as well as the resistance to crushing of timber across the grain, recommending the safe unit loads to be used. These were not, however, materially different from the safe loads already given in this volume.

MAXIMUM AIR-PRESSURE IN PNEUMATIC CAISSONS.

90. It has been stated that the limit of depth in the pneumatic process has been generally accepted as 100 ft. below the

water surface. The writer has also ventured to express the opinion that, with due care and reasonable precautions in selecting men and providing for their comfort and health, greater depths could be reached with safety. We have also seen that in the St. Louis and Memphis bridges the depth or immersion below the water surface reached 108 or 109 ft. Many lives were lost in the sinking of the St. Louis caissons. No report has been made on the other in regard to this point, so far as I have been able to find out. In the *Engineering News* of March 16, 1893, it is stated that a tunnel is now being constructed 8 × 10 ft. in cross-section, by the East River Gas Company, which will be half a mile long when completed. The headings of this tunnel are now 500 ft. out from the New York side and 100 ft. out from the Long Island end. The men have worked in compressed air at the respective depths of 134 ft. and 147 ft. below mean low tide. The men work in four-hour shifts. Thus far, one foreman has died and three workmen have been brought out unconscious. It is not stated what precautions, if any, have been taken for the health and comfort of the men.

Even as matters stand, the percentage of death and paralysis or unconsciousness would not seem to compare unfavorably with that in many preceding caissons when sunk to much less depths and requiring less intensity of air-pressure.

SOUNDINGS AND BORINGS.

91. The importance of accurate and thorough soundings or borings, in order to determine the character and lay of the strata underlying the bed of the river at any bridge site, has been earnestly urged in the first part of this volume. (See Art. 33, paragraphs 25, 26, 27.) In the *Engineering News* of April 13, 1893, is found a very instructive description of the removing and subsequent rebuilding of a pivot pier of a bridge over the Coosa River at Gadsden, Ala., which had badly settled on one side, "the pivot going down stream about 7 ft. and nearly throwing the swing-span into the river. The accident occurred at an extraordinarily high stage of water, the river being subject to a rise of nearly 40 ft. at this point." From the description and the accompanying drawings, it seems that there was a depth of 12 ft. of water at ordinary low stages, and 4 ft. of gravel, overlying the solid rock. The layer of gravel, it is stated, was "so compact, indeed, that it had proved impenetrable to the sounding-rod of the engineer originally in charge of the bridge, and was supposed by him to be solid rock." From this statement we are led to infer that the sounding was made with an ordinary straight rod, which was struck with a hammer or maul in order to determine the nature of the material at the bed of the river. That it is difficult, if not impracticable, to drive an inch or an inch and a half rod to any great depth in a compact bed of gravel or even sand is readily admitted; but that it would not penetrate at all, but would give a rebound and a sound easily recognized when a rod is simply lifted and dropped on solid rock, cannot be so readily admitted. If only one sounding was taken, the rod may have rested on a bowlder of large size; several soundings, however, would certainly have determined the question. It was certainly unwise to have sunk an open caisson, to simply rest on the bed of a river subject to such sudden and great floods, without a thorough examination of the nature of the bed. The driving of a single pile would have settled the doubt; and again, it would seldom be found that the bed of a river, if solid rock, would be sufficiently level to sink a solid bottom caisson on it without some careening. The bed being practically horizontal, should have at least aroused a suspicion that the bed was either gravel, sand, or clay. At the depth given—only twelve feet of water—it would seem that an ordinary coffer-dam should have been used, in which event the treacherous nature of the bed would have been discovered, a good foundation secured, and the subsequent danger and cost avoided. Had the soundings been made by the use of pipes and a force-pump the existence of the gravel bed would have been determined. (See Art. 33, paragraphs 25, 26, 27; and also Fig. 28.)

The height of the pier was about 80 ft., and it contained 1100 cu. yds. of masonry. As the pier had to be removed entirely, a large coffer-dam had to be built around the pier. "This dam gave a great deal of trouble during the prosecution of the work, *owing to the porosity of the gravel and to the irregularity of the surface of the rock upon which the gravel lay, although three separate rows of sheet-piling were driven through it to the rock* and well puddled between." Where sheet-piling could be so easily driven, an iron rod ought to have penetrated. The subsequent cost due to this error is not given; but the following estimate will be below rather than above the actual cost, and will, I hope, have the effect of strongly impressing upon young engineers the importance of obtaining reliable information, especially when it can be obtained at a cost of only $100 to $200 at the outside:

1100 cu. yds. masonry @ $8 = ...	$8,800 00
" " " " lifted from the pier, landed, cleaned, and piled, @ $3, ...	3,300 00
" " " " rehandled and relaid @ $3, ...	3,300 00
Constructing, pumping out of coffer-dam, and excavation, ...	5,000 00
Constructing false-work, trusses, etc., to support the draw-span, .	5,000 00
(Including in the above all material, labor, necessary plant, etc.)	
Total cost, ...	$25,400 00

The writer has made no estimates of actual quantities and costs, and hopes that he has not overestimated the costs. He hopes also that the engineer of this work will not consider the above as intended for a criticism of his skill or ability. We all make blunders, and the writer has recorded his own in several places in this volume; and he has taken every occasion to describe failures and blunders, with no other object in view than that of recording faithfully all facts exactly as they have occurred, as he felt it his duty to do; and he further believes that more useful knowledge and experience can be obtained from a study of blunders and failures than from successes.

The writer had a somewhat similar experience when building a bridge across the Warrior River in Alabama. He sounded with a solid-pointed rod, and reported rock at 5 ft. below the bed of the river, covered with sand and gravel. This result was somewhat anticipated, as the rock upon which stood an adjacent pier 100 ft. from the one under examination was well exposed at low-water. There were only some 6 or 8 ft. of water at lower stages at the site of the pier. He put in a coffer-dam, however, which developed the fact that there was at least 15 ft. of sand over the rock; and having pumped the water out and having excavated about 10 ft., it was then found necessary to drive piles in the bottom of the excavation to a further depth of 15 to 20 ft. in order to obtain a safe resistance.

One of our most eminent Southern engineers constructed a bridge across the Big Sandy River, W. Va., on a bed of bowlders and gravel at no very great depth below the bed of the river, which stood for many years without showing any settling, but after more than ten years of constant use one of these piers settled out of line. The writer called upon him for advice as to the suitableness of a bed of clay, under the Ohio River at Point Pleasant, W. Va., for supporting a high and heavy pier with its superstructure and load. His reply was, that when younger he thought that he knew a good foundation bed when he saw it, but after the settling of the Big Sandy pier he had come to the conclusion that he knew but little of this subject. Long experience will generally make us more cautious, and therefore safer advisers.

The engineer who has never blundered and never met with any failures has had but little experience or has acquired but little useful information, unless he is wise enough to profit by the experience of others, and has put himself to the trouble of acquainting himself with their failures.

THE ACTUAL RESISTANCE OF BEARING-PILES.

92. The importance of the subject of the bearing power of piles cannot be overestimated, and any information on the subject is valuable and instructive, and no excuse is needed for again referring to it. The following tables and remarks are taken from the columns of the *Engineering News* of Feb-

ruary 23, 1893. Outside of the valuable statistics given, the object of the article seems to be to prove that the formula

$$\text{Safe Load} = \frac{2wh}{s + 1}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

in which w = weight of hammer in tons or pounds (the safe load being in the same unit) and h = its fall in feet, s = set under last blow in inches, will give safe and reliable working loads. On this point the editor says:

"No formula can attempt to state exactly how much should be spent in such a case, or how much load can safely be placed on the pile. What the *Engineering News* formula does purport to do is to set a definite limit, high enough for all ordinary economic requirements, up to which there is no record of pile failures, excepting one or two dubious cases, where a hidden stratum of bad material lay beneath the pile, and above which there are instances of both excess and failure, with an increasing proportion of failures as the limit is exceeded.

"If it does this, as it is believed to do, it is in all cases a safe guide, having the risk of semi-fluid material existing beneath the foot of the pile, and in most cases a sufficient guide as well. But when a large number of piles are to be driven, or extra heavy loads are to be sustained, ordinary prudence would dictate the ascertaining by experiment just what the piles will bear, or, if failure would do no great harm, taking chances with greater loads without experiment, under favorable conditions. The formula is not intended to be rigidly applied to such cases as this." The above statement really contains all that the writer of this volume has contended for, when discussing the value of pile-driving formulæ.

The comparative merits of the *Engineering News* formula, Trautwine's formula, Crowell's modified formula, Sanders' formula, have been the subject of much discussion, heated and even acrimonious; and as the writer has taken little part in the battle of the formulæ, and is going to recommend the *Engineering News* formula, if any is necessary or used, no comparisons will be made, and the formulæ results given below refer only to this latter formula in the following records.

TABULATED RECORDS.

Table I.

Locality and Soil.	Hammer, Fall and Set.	Actual Load.	Safe Load by *Eng. News* Formula.
Chestnut Street Bridge......	1,200 lbs., 40 ft.		
Mud..................	$\frac{3}{4}$ in.	40,300	29,100 lbs.
Neuilly Bridge.............	2,000 lbs., 5 ft.		
Gravel....................	0.016 in.	105,300	19,700 "
Hull Docks.................	1,500 lbs., 24 ft.	45,000	
Mud....................	2 in.	to 56,000	24,000 "
Royal Border Bridge........	1,700 lbs., 16 ft.		
Sand and gravel..........	0.05 in.	156,800	53,700 "
Phila. Experiments..........	1,600 lbs., 36 ft.	14,560	
Soft mud................	18 in.	to 20,120	6,060 "
U. S. test-pile...............	810 lbs., 5 ft.		
Silt and clay	0.375 in.	59,600	6,600 "
French rule.................	1,344 lbs., 4 ft.		
	No set.	56,000	10,742 "

"The *Engineering News* formula gives the closest approximation of the three; and secondly, this formula for ultimate load (= six times the safe load) is not intended to be used for determining ultimate loads, and not alleged to give them with any accuracy, for the reason that the ultimate load is a much more variable quantity than the permanent safe load. At least we so understand it; and certainly the safe load is the only thing we are aiming to determine or care to know."

"It should be borne in mind that in some if not all of these cases the surrounding soil bears an unknown proportion of the load, so that the load actually coming on the piles may be several times less than stated."

Table II.

SUMMARY OF INSTANCES OF BEARING POWER OF PILES GIVEN IN MORE DETAIL BELOW.

Case No.	Actual Ultimate Load.	Safe Load by Formula.	Factor-of-Safety.	Material.
1.......	13,333	1,684	7.9	Mud
2.......	14,560	6,067	2.4	"
3.......	22,400	23,450	— 1.0	"
4.......	44,800	28,333	1.6	"

Case No.	Actual Ultimate Load.	Safe Load by Formula.	Factor-of-Safety.	Material.
5.......	15,175 to 47,375	11,400	1.3 to 4.1	Alluvial
6.......	59,618	6,741	8.8	Mixed
7.......	75,000	44,080	1.7	Sand
8.......	224,000	112,000	2.0	"
9.......	13,440	8,020	1.7	Sandy
		9,520	1.4	Mud
10.......	6,400 +	10,183	Not determinable.	"
	to 13,300 +	to 20,000	"	"
11.......	6,400 +	10,667	"	"
	to 13,333 +	to 11,790	"	"
12.......	over 22,400 +	37,500	"	"

TESTS OF PILES IN WHICH THE RESISTANCE TO EXTRACTION IS THE ONLY EVIDENCE AS TO ULTIMATE BEARING POWER.

Case No.	Actual Ultimate Load.	Safe Load by Formula.	Factor-of-Safety.	Material.
13.......	490	108		Sandy
	to 1,288	to 251		Clay
14.......	15,850	25,067		Unknown
15.......	25,000	5,000		Rotten rock
	to 50,000	5,000		
16.......	50,000	51,250		Sand
	to 83,000	to 88,000		"
17.......	71,300	12,800		Clay
	to 71,300	to 17,920		"

A detailed account of experiments for case No. 1 above has been given already. See Art. 42, par. 84.

Case 2. Philadelphia, 1873. Soft river mud. Trial pile loaded with 14,560 lbs. five hours after driving, and sank but a very small fraction of an inch. Under 20,160 lbs. it sank $\frac{3}{4}$ in.; under 33,600 lbs., sank 5 ft. (Records, Table I, show hammer weight 1600 lbs., falling 36 ft.; penetration or set 18 in. Safe load by formula, 6067 lbs.)

Case 3. Mississippi River at East St. Louis, 1868–69. Soft muddy bottom, with 5 or 6 ft. of water. Piles in temporary railway trestle of three-pile bent, 15 ft. from centre to centre, driven about 20 ft. Penetration, $2\frac{1}{2}$ to 3 in. The piles settled badly in a very short time under locomotives weighing not over 30 tons, so that the load on a pile could hardly have exceeded 22,400 lbs. (Safe load by formula,—taking $27\frac{1}{2}$ ft. mean fall, $2\frac{3}{4}$ ins. mean penetration, 1600 lbs. hammer,—23,450 lbs. Many of the data of this case are quite dubious, especially the weight given for locomotives. There were very few, if any, in Missouri in 1868–69 so light as 30 tons. It is more likely the load on each pile was double that stated.)

Case 4. Perth Amboy, 1873. Pretty fair mud, 30 ft. deep. Four piles, 12, 14, 15, and 18 ins. diameter at top, 6 to 8 in. at foot, were driven in a square to depths of from 33 to 35 ft. Distance apart not given. A platform was built upon the heads of the piles and loaded with 179,200 lbs., say 44,800 lbs. per pile. After a few days the load was removed. The 18-in. pile had not moved; the 12-in. pile had settled 3 in., and the 14 and 15 in. piles had settled to a less extent.

(Hammer 1700 lbs., falling 25 ft., with 2-in. penetration. Safe load by formula, 28,333 lbs.)

Case 5. The record of driving uncertain and unintelligible.

Case 6. Proctorsville, La. Material: mud, sand, and clay; wet. Trial pile (driven alone) said to have been 30 ft. long, yet it is said to have sunk 5 ft. 4 in. by its own weight, and to have been driven 29 ft. 6 in. deeper, making 34 ft. 10 in. driven length; cross-section $12\frac{1}{2} \times 12$ ins. at top, and $11\frac{1}{2} \times 11$ ins. sharpened to 4 ins. square at foot. Weight, 1611 lbs. Head capped. Pile bore 59,618 lbs. without settlement, but settled slowly under 62,500 lbs. Fall during last ten blows regulated to 5 ft. exactly. Penetration last ten blows ranged from $\frac{1}{4}$ to $\frac{1}{2}$ in.; mean, 0.35 in.; last blow, $\frac{3}{8}$ in.

(Hammer, 910 lbs.; safe load by formula, 6741 lbs., being very far below what the pile actually sustained. This is another case of those piles in soft material whose resistance is not fairly measured by the blows given when first driving, but can only be fairly gauged by trying blows after the mud has had time to set.)

Case 7. Buffalo. Material: wet sand and gravel. Piles driven in nests of from 9 to 13 piles. Test-pile of beech, 20 ft.

long after being driven and cut off. Driven length 20 ft., 3 ft. in stiff clay; cross-section, 15 ins. diameter at top. A load of 50,000 lbs. remained on the pile for 27 hours, but produced no appreciable effect. The load was increased 20,000 lbs. at a time, and left for 24 hours after each increase. A gradual settlement, aggregating $\frac{5}{8}$ in., took place under 75,000 lbs., and the pile then came to rest. The total settlement increased to $1\frac{1}{2}$ ins. under 100,000 lbs., and to $3\frac{1}{16}$ ins. under 150,000 lbs. During the experiments the ground was kept in a tremor by the action of three pile-drivers at work on the foundations. Subsequent use shows that 74,000 lbs. is a safe load.

(Hammer, 1900 lbs.; fall, 29 ft.; set, 1.5 in. Safe load by formula, 44,080 lbs.)

Case 8. As a result of the tests in Brooklyn, N. Y. Material: wet, loamy, micaceous quartz sand, becoming quicksand wherever it was much trodden. As the result of the tests it was believed that for a pile driven 33 ft. into the earth to the point of ultimate resistance, with a ram weighing 2240 lbs. and falling 30 ft. at the last blow, the extreme supporting power due to frictional surface was 224,000 lbs., or 1 ton per superficial foot of the area of its circumference.

(Safe load by formula for 0.0 and 0.2 in. penetration, 134,000 to 112,000 lbs.)

Case 9. Material: sand, with some mud. Piles seem to have been driven either by a steam pile-driver delivering 60 blows per minute, ram weighing 2205 lbs., falling 30 in., or by an ordinary hand engine, ram 992 lbs., fall 6 ft. 7 in. Penetration at last blow, $\frac{3}{4}$ in. to $1\frac{1}{8}$ in. If the penetration was over $\frac{3}{8}$ in. on the average of the last 100 blows, the rule was to put in extra piles. The load seems to have been about 13,440 lbs. per pile.

(Safe load by formula for 2205-lb. hammer, 8020 lbs.; with the 992-lb. hammer, safe load 9520 lbs.)

Case 10 belongs to the same set of experiments as Case 1, and is found fully described in part first of this volume.

Case 11 also belongs to Case 1. Safe load by formula, 10,667 to 11,790 lbs.

Case 12. Lake Pontchartrain Trestle, La. About 6 miles of trestle crossed the lake proper, and the remainder (16 miles) crossed the adjoining sea-swamp. Four-pile bents 15 ft. between centres. Material of swamp: several feet of soft, black vegetable mould, lying upon soft clay, with occasional strata of sand 1 to 2 ft. thick. Piles sank from 5 to 8 ft. of their own weight, and then about as much more with hammer (about 2500 lbs.) resting on head of pile. Two piles 65 ft. long were driven, one on the top of the other, and penetrated 9 in. with over 100 ft. driven; but a 30-foot fall, 30 minutes after driving a pile, gave only 3 in. penetration. Piles 65 to 75 ft. long; weight of ram about 2500 lbs.; fall about 30 ft.; penetration 3 to 12 in. "No settlement has been observed in the entire length of the structure to date." With four piles in a bent, and the bents 15 ft. centres, the load on each pile probably has not exceeded 22,400 lbs.

(Safe loads for 3, 6, 9, and 12 ins. penetration, 37,500, 21,430, 15,000, and 11,538 lbs. by formula. "As the only proper fall to be considered in a case like this is the 3-in. penetration, which occurred after 30 minutes' intermission, the check here is excellent.")

The remaining cases in Table 2, viz., 13, 14, 15, 16, and 17, only are important as enabling inferences to be drawn as to the bearing power of piles from their resistances to being pulled out after being driven. In Cases 13 and 14 the piles were lifted by the action of ice, and the load was estimated by the uplifting force of ice, which was taken at 18,850 lbs. per pile. "The case, in fact, is one of exceptionable doubtfulness in all respects."

Case 15. Material: wet, decomposed mica schist. A wrought-iron pipe, $3\frac{1}{2}$ ins. outside diameter, 3 ins. inside, was inserted in a bore-hole 6 in. in diameter and 30 ft. deep, and driven 14 ft. to rock. After several hours' work with block and fall, the pipe was pulled in two by using two hydraulic jacks of unequal power, one on each side, by means of which the pipe had been raised 8 ins. The fracture took place in the thread, where the wall thickness was reduced from $\frac{1}{4}$ to $\frac{1}{8}$ in., leaving a cross-sectional wall area of $3\frac{1}{8} \times 3.1416 \times \frac{1}{8} =$ 1.23 sq. ins.

Since the pipe had been slightly raised under the pull which soon after caused its rupture, the latter was evidently about equal to the resistance of the pipe to being withdrawn. We can estimate at the amount of this pull by estimating the tensile strength of the iron at the point of rupture.

The conditions of the experiment were very crude, but considering that the pipe was no doubt weakened by canting from side to side under the unequal forces exerted by the two jacks, as well as by repeated blows in driving and repeated tensile strains in drawing, and by the removal of the outer shell of the iron, in cutting the thread, the tensile strength could hardly have exceeded 40,000 lbs. per square inch, and, on the other hand, it was perhaps not less than 20,000, giving 25,000 to 50,000 lbs. as the extreme load.

(Hammer of 350 lbs. falling 8 ft., set $\frac{1}{8}$ in., sustained 25,000 to 50,000 lbs. Safe load by formula, 5000 lbs. scant.)

Case 16. Pensacola, Fla. Material: clean, hard, sharp, white quartz sand. All the sand would pass through a sieve having openings $\frac{1}{12}$ in. square. Water filtered through it came out perfectly clear. One cubic foot of it would hold 6 qts. of water. The 2-ton hammers could only drive it about 20 ft. The water was about $11\frac{1}{2}$ ft. deep. Seven piles, selected as representing the average of all, were tested with upward pulls of 20,000 lbs. each without moving, and one of these was afterwards tested with upward pulls sufficient to cause motion (as recorded below), and finally withdrawn. This pile was 29 ft. long, 16 ft. in sand, including its point—2 ft. long. One foot of this length was in loose sand, which had been excavated and had fallen back. The average diameter of the part in the sand was $13\frac{1}{2}$ ins., including the bark. Weight of pile 1632 lbs. Pile tested two months after driving. As neither the weight nor fall of hammer nor set are given, the formula can only be applied by assuming the necessary data.

				Safe Load by Formula.
1st..........	$W = 2,200$ lbs.,	$h = 30$ ft.,	$s = 0.5$,	88,000 lbs.
2d..........	$W = 4,100$ "	$h = 33$ "	$s = 0.0$,	270,000 "
3d..........	$W = 4,100$ "	$h = 10$ "	$s = 0.6$,	51,250 "

The tests on the trial pile resulted as follows:

78,000 lbs.........No movement.
80,000 " ....Resisted ½ min. and then rose very slowly. Rose 2¼ ins. in 30 minutes.
82,000 " ....1⅝ minutes.
83,000 " ....½ minute. Rose 2½ ins. in all in 30 minutes.
60,000 " ....18 hours. No movement.
64,000 " } ....Rose 3 ins. in one hour, 6 ins. in all.
74,000 " }
50,000 " ....For two days. No movement.

The very small loads obtained by the tests in this case seem to confirm the view already expressed, that the resistance of a pile to an upward pull must be less than that to a downward pressure, especially in a pure sand, exerting relatively a small resistance to being broken up, but offering a great resistance to a downward pressure.

Case 17. Albert Dock, Hull, England. Material: compact bluish clay, above which there were from 3 to 5 ft. of peat, and above this silt and sand in places. Piles of ordinary rough Memel bark timber, from 10 × 12 to 14 × 15 ins.; average 12½ ins. square, from 20 to 40 ft. long. Driven length from 6 to 30 ft.; average 18¼ ft. Most of the piles were driven from 10 to 20 ft. into the clay, and were nearly or quite in that material alone; but a few of the shorter piles, driven in a sloping side of the dock, were entirely in the silt, while a few others entered the peat without reaching the clay. The piles were driven close together in two rows 5 ft. apart forming a coffer-dam, the space between the two rows having been filled with puddled clay to above high-water mark. Before the piles were withdrawn the puddle was removed down to a level rather under high-water mark of ordinary neap tides. Weight of ram 2240 lbs., height of fall varied from 5 to 6 ft., and the penetration from 0.5 to 0.75 in. Four hundred and twenty piles were withdrawn and 300 observations recorded. The force was applied by men working a winch and estimated by testing that of the men in lifting known weights. The piles were drawn consecutively, so that one side of each pile was nearly or quite free from frictional contact, the opposite one was in loose contact with the adjoining pile, and only the remaining two sides resisted by friction with the ground.

The average total force required to draw a pile was 75,869 lbs. The author deducts from this 2340 lbs. ($= 12 \times 12$ in. $\times 15$ lbs. per square inch) for suction and 2240 lbs. for weight of pile, leaving, say, 71,300 lbs. as the frictional resistance to drawing the pile.

(Safe load by formula with $\frac{3}{4}$-in. set after 5-ft. fall, 12,800 lbs.; with $\frac{1}{2}$-in. set after 6-ft. fall, 17,920 lbs.)

The writer regards the above records as the most complete and satisfactory of any heretofore published, and although there is a noticeable want of uniformity and consistency in many respects, much can be learned from a careful study of all of the facts. The methods of testing are suggestive, and the importance of accurate records are clearly set forth, and whereas there are great differences between the actual loads and the loads obtained by the formula, a comparison of these would seem to lead to the conclusion that what is claimed by the authors of the formula, viz., that the formulæ will usually err on the safe side, can be reasonably admitted.

As was mentioned in another place, it is but reasonable to suppose that it would require a smaller force to pull a pile than it would to cause it to sink; as in the first case when the pile is lifted a gradually decreasing surface is in contact with the surrounding soil, and in fact in the stiffer soils it is only necessary to move the pile but a few feet upward before the frictional resistance will disappear almost entirely if the lifting force is accurately in the prolongation of the axis of the pile; whereas in sinking a pile by direct pressure, gradually increasing surfaces are presented in contact with the soil, so that not only has frictional resistance to be overcome, but a certain amount of lateral and downward compression of the material has to be effected by the required displacement of the material. This explains the fact that structures founded on piles often settle a few inches, and then remain fixed for a time, when a small additional settlement may take place.

PRESERVATION OF TIMBER—VULCANIZED TIMBER.

93. The great importance of increasing the durability of timber, and of devising some means by which many kinds of timber which are now considered useless can be utilized, has been realized by a few engineers and builders, but has not received as much recognition as the subject demands. Therefore the writer wishes to give some prominence to a process which is not generally known, by which it is claimed all woods are increased in strength and stiffness, are made more durable, and that many kinds of wood now considered worthless can be utilized in building any kind of timber structures. Several methods have been given in the preceding pages of preserving timber (see Article 38). It has not, I believe, been claimed that any of these methods increased the strength of timber, or even that to any extent it rendered the weaker and inferior grades suitable for the ordinary structures.

The method now to be described seems to the writer to be rational, simple, and economical, and if the results thus far obtained are confirmed by future experiments, and the processes can be economically carried out, it would seem that the question of timber preservation will have been satisfactorily solved. An interesting description of the method of producing so-called "vulcanized timber" will be found in the *Electrical World*, March 11, 1893. From this magazine the following points of interest are mainly taken.

Wood as it occurs in nature consists of cellulose impregnated with resin, volatile oils, sugar, gum, tannin, protein bodies, and the usual mineral constituents of plants. When wood is heated, as in ordinary distillation, the cellulose decomposes and a chemical change takes place between it and the natural constituents of the sap, resulting in a most powerful antiseptic mixture containing acetic acid, methyl alcohol, acetone, methyl acetate, tarry matter containing phenol, creosote, carbolic acid, and about thirty other chemicals of lesser practical importance. These chemicals and antiseptics result from the action of heat on the natural sap of the wood, and are entirely different from the original sap, which allows the attacks of microscopic fungi and decay. If timber is heated to the temperature which will produce the above change and the antiseptic mixture is kept in it by pressure, instead of distilling it out, experiments show that the change will produce a stronger and more durable timber than it was originally. "Wood-vulcanizing is heating wood and timber under great pressure." The wood is heated in closed cylinders from eight to twelve hours, at a temperature ranging from 300° to 500° Fahr., while under a pressure of 150 to 200 lbs. per square inch. A circulation of superheated and dried compressed air removes the surface moisture and any water that does not take part in the reaction, and combine with the woody constituents. Cylinders of steel $105 \times 6\frac{1}{2}$ ft. in length and diameter, respectively, are employed. The timbers to be treated can be loaded on cars or trucks and run direct into the cylinders. When subjected to the proper temperature and pressure during the necessary interval of time, the timber is removed. "This apparently makes decay impossible by sealing up the pores with antiseptic matter, which becomes solid on cooling. The changed sap is very dark or black."

"The process of vulcanizing seasons all timber, preventing any further warping, checking, or cracking. Such timber is not influenced by atmospheric agencies, bacteria or spores, and requires no paint for protection. The albuminous constituents of the natural wood have been coagulated by the high heating, and rendered insoluble."

These are seemingly extravagant claims, which should only be accepted when fully established by careful and accurate experiments in sufficient numbers and after a sufficient lapse of time—at least so far as durability is concerned. But when so established they should be accepted by engineers, whether in conflict with theories or not, and even when seeming to be unreasonable. Some chemists have stated to the writer that it seems probable that the chemical changes could take place only when such a temperature was reached that would char and destroy the woody fibre ; so theoretically the results seem doubtful. But they at least seem reasonable in view of the facts: 1st. That timber is seasoned artificially by being subjected to hot air for comparatively short periods of time. 2d. That timber is rendered more durable and capable of resisting the attacks of the teredo when the moisture has been removed and the pores refilled with some at least of the above-mentioned antiseptic compounds by pressure. 3d. It would seem to be more economical to convert the fluids found in the timber into antiseptics and keep these in the timber by similar processes to those adopted when creosote is forced under pressure into the timber, the creosote being first obtained by distillation from other and different timber.

However, to sustain the claims of the inventors or users of the vulcanized process for preserving timbers, they offer the following results of experiments and experience. And when we remember the fact that the annual consumption of timber in this country, equals twice the amount of material supplied by the annual growth of our forests, it is to be hoped that an efficient and economical means of preserving, strengthening, and hardening timber has been discovered.

Experiments made on a number of specimens show that the strength is increased as much as 18.78 per cent, and amount of deflection is decreased by 13 per cent, as compared with specimens of the same timber that have not been treated.

Timber not treated by the vulcanizing process, but painted, showed a loss of 38.12 per cent in strength as compared with the treated timber, after a lapse of some considerable time in exposed situations. Frames made partly of the vulcanized timber and partly of the timber in its natural state, after the lapse of eight years or more were found sound and solid so far as the prepared timber was concerned, but those parts formed of the natural timber had almost entirely rotted. Mr. Tracy, late Secretary of the Navy, made a thorough investigation of the subject, and recommended that the vulcanized timber should be used in certain parts of the ships being constructed for the Government. These facts would seem to justify, in part at least, the claims of the manufacturers.

TENSILE STRENGTH OF CEMENTS.

REQUIREMENTS.

	Tensile Strength required in Pounds per square inch. One day.	One week.
Portland cement, neat	110	300
" " 1 to 2 mortar		100

Fineness: 80 per cent must pass through a sieve of 10,000 meshes.

AVERAGE TENSILE STRENGTH IN POUNDS.

Time of set in water	1 day.	1 week.	1 mon.	1 year.	2 years.	3 years.	4 years.
Portland, Burham (neat)	167	429	615	798	700	764	782
" 1 to 2 mortar	...	141	258	468	532	632	658
" 1 to 3 "	...	169	224	404	520	552	...
Giant (neat)	140	348	422	682	694	736	771
" 1 to 2 mortar	...	166	280	490	564	680	674
1 to 3 "	...	140	234	420	512	572	...
Rosendale, Union (neat)	100	240	228	510	542	650	654
" 1 to 2 mortar	...	34	94	394	430	514	522

A natural Portland cement from Maria Island, Tasmania, weighs 113 lbs. per bushel. Specific gravity 3.152; slow setting.

	7 days.	14 days.	28 days.
Tensile strength (neat)	536	536	646
" " 1 to 3 mortar	150	...	258

The above is taken from *Eng. News*, April 13, 1893. It is of special interest, as the tables contain the longest time tests of which we have any records, and, extending through a period of four years, give the gradual increase in hardness and strength with increase of age. The fall in strength between the one- and two-year test of the Burham neat cement is due either to a typographical error or an error in the record; the one-year test should no doubt be 698 lbs. The results are, however, recorded as found in the columns of the *News*. There is also apparently an error in the one-week test of same cement in the 1 and 2 or the 1 and 3 mortar.

These tables are given, as interesting and instructive results of experiments, without comment.

EXTRACTS FROM THE BUILDING ORDINANCES OF THE CITY OF CHICAGO.

Class 1st. Buildings devoted to the sale, storage, or manufacture of merchandise, and stables.

Class 2d. Residences for three or more families, hotels, boarding or lodging houses occupied by twenty-five or more persons, and office buildings.

Class 3d. Residences for one or two families, or for less than twenty-five persons.

Class 4th. Buildings used as assembly halls for large gatherings of people.

Fire-proof construction applies to buildings in which all parts that carry weights, stairs, elevator enclosures and their contents are made of incombustible material, and in which all metallic structural members are protected against the effects of fire by coverings of an incombustible and slow heat-conducting material, such as brick, hollow tiles or burnt clay, porous terra-cotta, and two layers of plastering on metal lath.

In "skeleton construction" all external and internal loads and strains are transmitted from the top of the building to the foundations by a skeleton or framework of metal. In columns of rolled iron or steel, the different parts shall be riveted to each other, and shall be united by riveted connections to the beams and girders resting upon them. In cast-iron columns, each successive column shall be bolted to the one below it by at least three $\frac{5}{8}$-in. bolts, and the beams and girders shall be bolted to the columns.

Slow-burning construction applies to all buildings in which the structural members are made wholly or in part of combustible material, but throughout which all materials shall be protected against injury from fire by coverings of incombustible, slow heat-conducting materials, such as above described. Oak posts of greater sectional area than 100 sq. in. need not have any special fireproof covering.

"Mill construction" applies to buildings in which all of the girders and joists supporting floors and roofs have a sectional area of not less than 72 sq. in., and above the joists of which there is laid a solid timber floor not less than $3\frac{3}{4}$ in. thick. Wooden posts in buildings of this class are to have an area of at least 100 sq. in. Iron columns, girders, or beams must be protected as provided for fire-proof buildings, but the wooden posts, girders, and joists need not be covered.

Ordinary construction applies to those buildings in which the timber and iron structural parts are not protected with fire-resisting coverings.

Buildings of classes 1 to 3 shall be made entirely of fire-proof construction, if 100 feet or more in height, and entirely of slow-burning or of mill construction if between 60 and 100 ft. in height. They may be built of ordinary construction if under 60 ft. in height. Buildings of class 4, containing not more than 600 seats, may be built of ordinary construction; if containing from 600 to 1500 seats, they shall be built of slow-burning or of mill construction, and if more than 1500 seats, of fire-proof construction entirely. If movable scenery is used, they must be of slow-burning or mill construction when containing less than 1000 seats, and entirely fire-proof if containing more than 1000 seats.

No building shall be erected of greater height than 130 ft. from the sidewalk level to the highest point of external bearing walls. The height of no building of skeleton construction shall be more than three times, and no isolated building of masonry construction more than four times, its least horizontal dimension.

Loads.—Buildings of class 1 shall be designed for a minimum load of 150 lbs. for each square foot of floor surface. For buildings of the classes 2 and 4, a live load of 70 lbs. per square foot shall be assumed in addition to all permanent loads. In determining the strength of posts and the area of foundation for many storied buildings of classes 2, 3, and 4, allowances are to be made for the fact that the before-mentioned live load of 70 lbs. per square foot is but an occasional load, which rarely occurs simultaneously upon corresponding parts of many floors, and, if so, for a brief period only.

Foundations shall be constructed of either of the following

Cement, concrete, dimension or rubble stones, sewer or paving bricks, timber piles covered with a grillage of oak timber, or an oak grillage may be used, all timber to be below city datum. Rails or beams used as parts of foundations must be thoroughly imbedded in concrete, and around the exposed external surfaces of such concrete foundations there must be a coating of cement mortar at least 1 in. thick. In pile foundations the piles shall be driven to reach the underlying stratum of hard clay or rock, and shall not be loaded with more than 25 tons. In other but pile foundations the limits of loads for different kinds of soil are to be the following;

If the soil is a layer of pure clay not less than 15 ft. thick, without admixture of any foreign substances, excepting gravel, 3500 lbs. per square foot. If the soil is a layer of dry sand, 15 ft. or more in thickness and without admixture of clay, loam, or other foreign substance, 4000 lbs. per square foot.

If the soil is a mixture of clay and sand, 3000 lbs. per sq. ft.

The offsets of foundations of concrete alone shall not exceed one half the height of the respective courses, and such concrete foundations must not be loaded more than 8000 lbs. per square foot. If reinforced by rails or beams, the offsets must be so adjusted that the fibre strain per square inch shall not exceed 12,000 lbs. for iron nor 16,000 lbs. for steel.

Dimension stones must have uniform beds, and offsets of layers must not be more than three quarters of the height of the individual stones. The load per square foot in foundation piers of dimension stones shall not be more than 10,000 lbs.

In brick piers there shall be at every offset a bond-stone at least 8 in. thick, and at the top of each pier a cap-stone at least 10 in. thick, or in all such cases a bond-plate of cast or rolled iron. The extreme loads on brick-work laid in mortar of any cement, established as a standard by the Society of Civil Engineers of the Northwest, are 25,000 lbs. per square foot, and 18,000 lbs. for brick-work laid in ordinary cement mortar. The use of soft bricks for piers is prohibited.

Maximum Permissible Stresses.—Cast-iron crushing stress for plates, 15,000 lbs. per square inch; for lintels, brackets, or corbels, compression 13,500 lbs. per square inch and tension 3000 lbs. per square inch; for girders, beams, corbels, brackets, and trusses per square inch, 16,000 lbs. for steel and 12,000 lbs. for iron.

For plate girders:

$$\text{Flange area} = \frac{\text{max. bending moment in ft.-lbs.}}{CD},$$

in which D = distance between centres of gravity of flanges in feet;
C = 13,500 for steel and 10,000 for iron.

$$\text{Web area} = \frac{\text{max. shear}}{C},$$

where C = 10,000 for steel and 6000 for iron.

For rivets in single shear per square inch of rivet area:
If shop driven, 9000 lbs. for steel and 7500 lbs. for iron.
" field " 7500 " " " " 6000 " " "

Maximum Permissible Load.

For cast-iron round columns:

$$S = \frac{10000 \,.\, a}{1 + \frac{l^2}{600d^2}}. \left\{ \begin{array}{l} a = \text{area of column in square inches;} \\ l = \text{length of column in inches;} \\ d = \text{diameter} \quad \text{"} \quad \text{"} \quad \text{"} \end{array} \right.$$

For cast-iron rectangular columns:

$$S = \frac{10000 \,.\, a}{1 + \frac{l^2}{800d^2}} \left\{ \begin{array}{l} l \text{ and } a \text{ as above;} \\ d = \text{least horizontal dimension of column.} \end{array} \right.$$

For riveted or other forms of wrought-iron columns:

$$S = \frac{12000 \,.\, a}{1 + \frac{l^2}{36000r^2}}. \left\{ \begin{array}{l} l \text{ and } a \text{ as before;} \\ r = \text{least radius of gyration in inches.} \end{array} \right.$$

For riveted or other steel columns if more than $60r$ in length:

$$S = \left(17000 - \frac{60l}{r}\right)a.$$

For riveted or other steel columns if less than $60r$ in length:

$$S = 13500 . a.$$

l, r, and a as in preceding examples.

For timber girders:

$$S = \frac{cbd^2}{l}. \left\{ \begin{array}{l} l = \text{length of beam in feet;} \\ b = \text{breadth of `` `` inches;} \\ d = \text{depth `` `` `` ``} \\ c = \text{160 for long-leaf yellow pine;} \\ \quad = \text{120 for oak;} \\ \quad = \text{100 for white or Norway pine.} \end{array} \right.$$

For timber posts:

$$S = \frac{ac}{1 + \frac{l^2}{250d^2}}. \left\{ \begin{array}{l} a = \text{area of post in square inches;} \\ d = \text{least side of rectangular post in inches;} \\ l = \text{length of post in inches.} \\ c = \text{600 for white or Norway pine;} \\ \quad = \text{800 for oak;} \\ \quad = \text{900 for long-leaf yellow pine.} \end{array} \right.$$

Wind-bracing.—In all buildings the height of which is more than one and one half times their least horizontal dimension, allowances shall be made for a wind-pressure of not less than 30 lbs. for each square foot of exposed surface. As factors of resistance to wind-pressure may be counted: 1st. Dead weight of structure, especially in its lower parts; 2d. Diagonal braces; 3d. Rigidity of construction between vertical and horizontal members; 4th. Construction of iron or steel columns in such manner as to pass through two stories with joints breaking in alternate stories.

Towers, domes, and spires may be built on the top of the roofs of buildings in classes 1, 2, and 3, but shall not occupy more than one quarter of the street frontage of any building or have a base area of more than 1600 square feet. If built to heights between 60 and 90 feet above the sidewalk, they shall be of slow-burning construction, and if more than 90 feet above sidewalk of fire-proof construction. Where the area of such spire, dome, or tower exceeds 100 square feet its supports shall be carried down to the ground and shall be of slow-burning construction if the supported structure is between 60 and 90 feet high, and of fire-proof construction if more than 90 feet high.

SOME RECENT CONTRACT PRICES ON RAILWAY MASONRY.

For rock-faced ashlar masonry......	$8.00	per cubic yard.
" broken range masonry.... ...	6.00	" " "
" random rubble "	4.50	" " "
" ashlar arch "	10.00	" " "
" rubble " "	7.00	" " "
" rectangular culvert masonry....	3.00	" " "
" paving, dry..................	1.50	" " "
" " , cemented.............	2.50	" " "
" vertical wall.................	2.50	" " "
" slope "	2.00	" " "
" riprap.	1.00	" " "
" white-oak timber.............	30.00	per 1000 feet B. M.

These would seem to be bottom prices for the several classes of work, and are those actually paid on railway work in 1892.

EXPERIMENT ON THE BEARING POWER OF PILES.

(Taken from *Eng. News*, July 6, 1893.)

As it was desired to erect the Chicago Public Library building on a pile foundation constructed by driving three rows of piles in a trench, and to determine whether it would be safe to load each pile with 30 tons, the following experiment was made on a group of four of the piles; and in order to make the experiment under the same conditions as would exist under the structure, three rows of piles were driven into the trench, the piles in the middle row being then cut off below the level at which those in the outside rows were cut off, so as to bring the bearing only on four piles, two in each outside row. This gave the benefit arising from the consolidation of the material by the other piles. Fifteen-inch steel **I**-beams were then placed on the piles, and upon these a platform 7 feet by 7 feet composed of 12 × 12-inch yellow-pine timber.

The piles were driven by a steam hammer of the Nasmyth type, weight 4500 pounds, fall 42 inches, making 54 blows per minute. The last 20 feet were driven with a follower of oak. To drive the last foot required 48 to 64 blows: it may be estimated that without using the follower it would have required 24 to 32 blows to have driven it the distance of a foot. In the same soil it required about 16 blows of a drop-hammer weighing 3000 pounds and falling 30 feet to drive the last foot with a follower, and 32 to 36 blows of the same drop-hammer falling 15 feet with a follower. The piles were driven $2\frac{1}{2}$ feet between centres, three in a row along the trench. They were 54 feet long and were driven about $52\frac{1}{2}$ feet—about 27 feet in soft, plastic clay, 23 feet in tough, compact clay, and 2 feet in hardpan. They had an average diameter of 13 inches, circumference of 41 inches, and area at small end of 80 square inches. It has been found that similar piles, after being driven and allowed to stand for 24 hours, required from 300 to 600 blows of the above-described hammer to drive it the last foot, or a repetition of 300 to 600 blows of 180,000 inch-pounds each. The heads of the piles were sawed off 27 feet below the street grade, the lower ends being about 80 feet below same. The bearing power of this hardpan by Rankine's formula may be taken at 170 pounds per square inch, and by empirical results at 250 pounds per square inch; and at a fair assumption it may be taken at 200 pounds per square inch. The extreme average frictional resistance per square inch of sides of piles like those described, as deduced by experiments made under analogous conditions, may be taken at 15 pounds per square inch. The area at lower end beirg 80 square inches, and the bearing resistance of the hard pan 200 pounds per square inch, the extreme point resistance will be 16,000 pounds. The average total exterior surface of one pile will be about 25,000 ($52 \times 12 \times 41 = 25,584$) square inches, which at 15 pounds will give a total frictional resistance of 375,000 pounds or an ultimate bearing power of 195.5 tons, or, discarding point resistance, of about 187 tons. And assuming that the ultimate crushing strength of wet Norway pine is not over 1600 pounds per square inch, and with a safety factor of 3, the safe load will not be over 533 pounds per square inch; and as the piles have a minimum diameter at lower end of 8 inches and at the butt 16 inches, the minimum average area will be about 113 square inches; hence each pile should not have to carry over 60,230 pounds, or 30 tons about. This, then, provides a factor of safety of 3 for the crushing resistance of the timber, and a safety factor of 6 for the frictional resistance of the soil. If the timber be loaded to one half of its ultimate strength, 90,000 pounds or 45 net tons may be assigned to each pile. But in this case only 30 tons was allowed. The experimental test was made by piling pig-iron on the platform resting on four of the piles, which were five feet centres, the entire platform being 7 7 feet, as before stated. The pig-iron was piled up at irregular intervals. When four feet high the load was 45,200 pounds, and was then continued until at the end of about four days it was 21 feet high, giving a load of 224,500 pounds. Levels were taken, but no settlement had occured. By the end of about eleven days the pile of iron had reached the height of 38 feet, giving a load of 404,800 pounds upon the four piles, or about 50.7 tons per pile. Levels were then taken at intervals during a period of about two weeks, and no settlement having been observed, a load of 30 tons was considered perfectly safe. The iron was removed, and the construction of the building was commenced.

FOUNDATIONS FOR HIGH BUILDINGS.

As stated in other parts of this volume, some of the high buildings in Chicago and other cities rest on a foundation of iron or steel beams and concrete, so spreading the base that the weight transmitted through iron columns will not exert more than a safe unit strèss of from 3000 to 4000 lbs. per square foot of the clay foundation-bed; others are built on piles of wood driven to the hard-pan or rock underlying the clay, at a depth of from 50 to 100 ft., notably the Chicago Stock Exchange Building. Near adjacent buildings shafts 5 ft. in diameter, lined with oak poling-boards 4 ft. long and braced with heavy iron hoops, were employed in order to avoid any possible chance of causing cracks or settlements as a result of vibrations due to driving wooden piles. These shafts were sunk to a depth of 55 ft. below bottom of excavation. A pile foundation was also adopted under the Public Library. The piles, about 54 ft. in length, were driven to a depth of 52½ ft. (27 ft. in soft clay and 23 ft. in compact clay), using a Vulcan-Nasmyth steam-hammer. There was no settlement under loads of 50 tons per pile, the load remaining on the pile for 11 days. The number of piles was determined by the condition that no pile should have to carry a load exceeding 30 tons. Under each column of the Fisher Building a group of 25 pilės was driven to consolidate the wet clay. A 6-in. bed of concrete was packed around the heads of the piles, and on this rested a foundation of steel beams and concrete of the usual construction. For the foundation of the recently constructed Post-office, wooden piles 50 ft. in length were driven in the bottom of an excavation of 28 ft. in depth.

In the latest development of foundations for high buildings in Chicago, the well or shaft system is being largely adopted. Shafts are sunk until the hard-pan or solid rock is reached, and the bottom of the shaft may or may not be enlarged; the shaft for its full length is then filled with concrete. These shafts are sunk as open wells, lined with vertical tongued and grooved poling-boards, and braced on the inside with iron hoops, which are made in semicircular forms with ends bent inwards, in order to form flanges. In some cases steel cylinders are sunk instead of the wooden lining in open shafts. These are necessary or desirable where layers of quicksand are encountered. Under the Edison Building, covering an area of 75×90 ft., a layer of quicksand was found, at a depth of about 100 ft. below the street-grade, 10 to 12 ft. in thickness. This quicksand was underlaid with boulders from 4 to 5 ft. in diameter. In this case the wood-lined shaft was used until the quicksand was reached; a steel cylinder, made in three sections, with vertical joints and angle-iron connections, was then put down. As the quicksand was removed, the cylinder settled by its own weight until it rested on the boulders, which had to be drilled and split. There were twenty-four cylinders, eighteen 6′ 6″ and six 8′ 6″ in diameter. Jacks had to be used to force the cylinders down to bedrock after removing the boulders.

The open-caisson or shaft type of foundations is being adopted to a large extent. The diameter of the cylinders depends upon the load which they have to support. The smallest are about 3 ft. 2 in., and the largest are about 10 ft. in diameter. The specifications require the shafts to be sunk to hard-pan, about 70 to 90 ft. below city datum, or to solid rock, about 90 to 110 ft. Where the excavation stops short of solid rock, the shaft is enlarged at the bottom to twice the diameter of the shaft proper, with side slopes at about 1 to 1. The excavation is usually made by hand and the material lifted out in buckets, which are raised by windlass operated by hand- or steam-power. The usual method of sinking shafts is adopted. An excavation is made to a depth of 4 ft., with a diameter 4 ins. in excess of the size of concrete column required; the 2-in. lagging for lining is placed, then three interior rings or hoops are inserted in order to hold the lagging, after which another section of 4 ft. is excavated and lined. In this manner the excavation is made to the full depth required. The lagging is generally strips of pine 4 ft. $\times$ 6 ins. $\times$ 2 (or 3) ins. In a soft or flowing material, such as underlies the city of Chicago, it is important not to make the excavation larger than necessary, as the material will flow or be squeezed in to fill any empty space around the lagging, and may result in damage to adjacent structures even where a considerable space intervenes between the structures.

The concrete filling is generally composed of 1 part high-grade Portland cement, 2 parts clean, sharp sand, and 4 parts broken stone. The concrete may be mixed by hand, though more commonly in some kind of mechanical concrete-mixing machine. It is simply dumped in the hollow space, lowered in buckets, or allowed to slide down through chutes, as may be required. It is well tamped in all cases. It requires from 10 to 12 days to complete one column, working night and day. The specified load varies from 35,000 to 50,000 lbs. per square foot of column area.

Figs. 63, 64, 65, and 66 fully explain the shaft, or well, method of securing foundations for high buildings and the con-

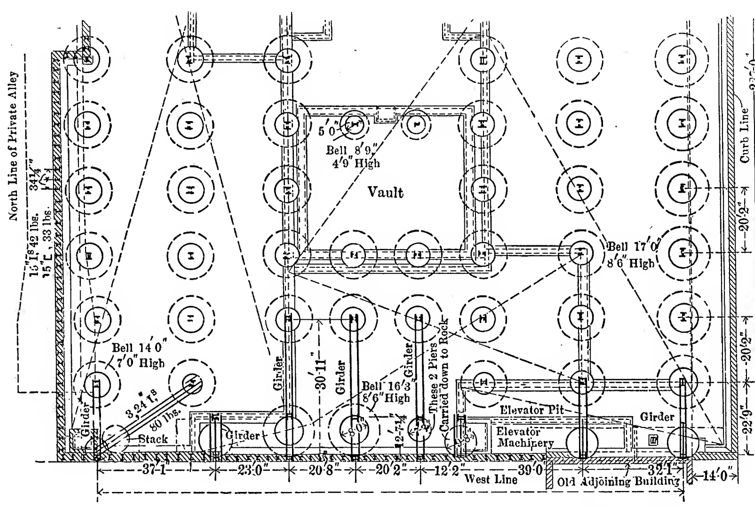


FIG. 63.—PLAN OF FOUNDATION PIERS OF FIRST NATIONAL BANK BUILDING, CHICAGO.
(D. H. Burnham & Co., Architects.)

struction under the sidewalks. For further description and discussion, see *Eng. News*, Dec. 22, 1904.

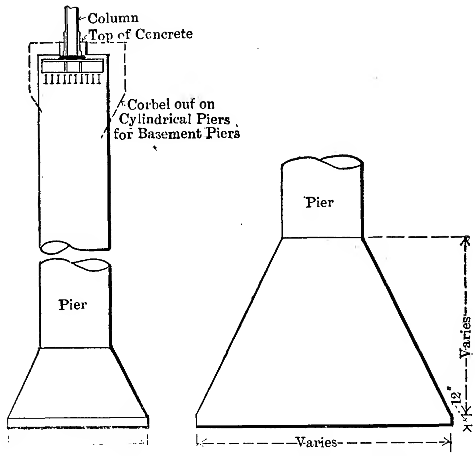


FIG. 64.—FOOTINGS AND CAPPING OF CONCRETE PIERS; FIRST NATIONAL BANK BUILDING, CHICAGO.

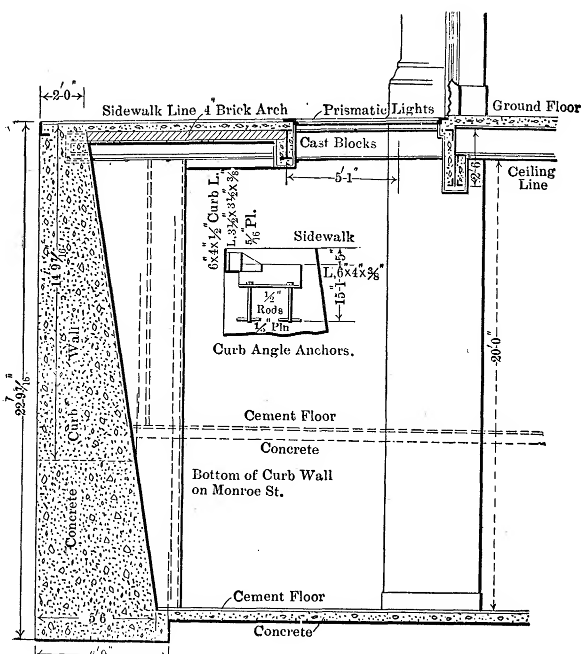


FIG. 65.—SECTION OF CURB RETAINING-WALL.

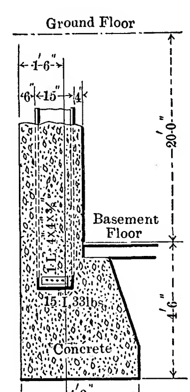


FIG. 66.—RETAINING-WAL

CONCRETE-STEEL RAILROAD BRIDGE AT PLANO, ILLINOIS.

THIS arch is for double track, 44 ft. wide, and span of arch 75 ft. Long concrete wing-walls for the abutments bring the total length of structure to 212 ft.

The arch is of the three-centre type; the intrados at and on either side of the crown has a radius of 43 ft., and of 20 ft. for the haunches. The radius of the extrados is 59 ft. The thickness at the crown is 3 ft. The arch was designed to have sufficient thickness for strength without the reinforcing steel bars. Any strength arising from the steel bars is in excess of, and additional to, the computed requirements. The reinforcement consists of $\frac{7}{8}$-in. Johnson corrugated steel bars, 12 ins. apart, each row having four bars, 4 ins. and 6 ins. from the intrados and extrados, respectively. The bars of each pair are staggered to give a spacing of 6 ins., and between them are placed $\frac{3}{4}$-in transverse bars about 2 ft. apart. For full details see *Eng. News*, Dec. 22, 1904. The concrete of the arch is composed of 1 part Portland cement, 2 sand, and 4 broken stone; that for the foundations, spandrels, and abutments, of 1 cement, 3 sand, and 6 stone and gravel. To prevent freezing, the concrete mortar was subjected to a steam-jet, which played on the concrete until an initial set had taken place.

RETEMPERING CEMENT MORTAR.

IN *Eng. News*, July 24, 1902, is found an interesting article on some experiments with cement, mortar, and concrete, by Thomas S. Clark. From a number of tests he found that the strength of mortar made with Rosendale cements was seriously impaired by retempering; on the contrary, tests with Portland cement resulted in no injury to strength when retempered an hour or more after first mixing. A few examples from a table show these results:

ROSENDALE-CEMENT MORTAR—NEAT CEMENT, WATER 28 P. C.

Hours in Air.	Age in Days.	Tensile Strength.		Remarks.
		Average Number of Tests.	Pounds per Square Inch.	
3	1	43	139	Not retempered.
3	1	13	27	Retempered after 60 minutes.
24	28	6	219	Not retempered.
24	28	6	94	Retempered after 60 minutes.
24	56	5	322	Not retempered.
24	56	3	180	Retempered after 60 minutes.
PORTLAND-CEMENT MORTAR—NEAT CEMENT, WATER 18 P. C.				
3	1	54	225	Not retempered.
3	1	17	115	Retempered after 60 minutes.
24	7	42	730	Not retempered.
24	7	20	779	Retempered after 60 minutes.
24	30	15	816	Not retempered.
24	30	16	888	Retempered after 60 minutes.
ROSENDALE CEMENT—1 CEMENT, 3 SAND MORTAR, WATER 14 P. C.				
24	28	5	39	Not retempered.
24	28	5	31	Retempered after 60 minutes.
24	56	5	73	Not retempered.
24	56	5	59	Retempered after 60 minutes.
24	112	5	113	Not retempered.
24	112	5	56	Retempered after 60 minutes.
PORTLAND CEMENT—1 CEMENT, 3 SAND MORTAR, WATER 10 P. C.				
48	28	5	254	Not retempered.
48	28	5	280	Retempered after 60 minutes.
48	90	5	280	Not retempered.
48	90	5	292	Retempered after 60 minutes.
48	180	5	327	Not retempered.
48	180	5	357	Retempered after 60 minutes.

The writer thinks that after the chemical process of setting has reached a certain stage, any disturbances of the mass will kill the hardening process; that this occurs in a shorter time with Rosendale cement than with Portland cement, and that retempering Portland-cement mortar after having been mixed for two hours or more will result in injury to the mortar. It is clearly good practice to use mortar promptly after mixing, though not always essential to secure good results.

Moulding Briquettes.—It seems to be better to place the mortar in moulds with only sufficient pressure to fill them thoroughly. In order to use much pressure it is necessary to mix the mortar rather dry. This results in briquettes giving irregular results and less indicated strength than with more water.

NEAT CEMENT IN AIR 24 HOURS.

Age, in Days.	Per cent of Water.	Tensile Strength, Lbs. per Sq. In.
7	18	722
7	20	680
7	22	638
28	18	684
28	20	762
28	22	809

There is a certain quantity of water required for the full chemical and mechanical action in setting. If this amount of water is not present, no amount of force or pressure will give the strength that will result from a sufficiency of water. The amount of water varies with the brand or character of cement. For this reason, especially in hot weather, it is better to have the concrete rather wet. If forced to lay Portland-cement concrete in freezing weather, the least possible quantity of water should be employed.

The following table gives some results with sand and stone-dust and Portland cement, with 10 p. c. water, the specimens having been exposed 24 hours in air, 6 days in water:

Cement.	Sand.	Stone-dust.	Tensile Strength, Lbs. per Sq. In.
1	..	2	245
1	2	..	345
1	..	3	216
1	3	..	241

The explanation of the results in the above table is that a fine sand or stone-dust requires more cement than a coarser material, ordinary sand, and with the same proportions the stone-dust gives the weaker mortar.

Crushing Strength of Concrete using Selected and Unselected Stone.—The results of tests to ascertain the crushing strength of concrete with both selected and unselected stone are given below. The unselected stone contained some mica and a small per cent of quartz; the selected stone was standard limestone broken to sizes of 1 to 2 ins. in longest dimension. The stone containing a large per cent of mica was rejected, the mica having so smooth a surface that good bond and adhesion of mortar would be reduced.

Twelve 8-in. cubes, mixed in the proportions of 1 Portland cement, 3 sand, and 5 stone, were made. These cubes were kept in air 24 hours and in water 5 months, then tested in compression. The table gives averages of two specimens. In the first set fine sand was used with the following results:

(1)	1595	lbs. per	square	inch	compression,	standard	limestone
(2)	1185	"	"	"	"	selected	stone
(3)	985	"	"	"	"	rejected	"

With coarse, sharp sand the results were:

(1)	1825	lbs. per	square	inch	compression,	standard	limestone
(2)	2145	"	"	"	"	selected	stone
(3)	1102	"	"	"	"	rejected	"

A number of cubes and beams were made from concrete as mixed and ready for use in the work. Cubes of 8 ins. on side, one day in air and 40 days in water, showed a compressive strength of 1765 lbs. per square inch. A concrete beam $10 \times 10 \times 60$ inches, 72 hours in air and 39 days in water, clear span 54 ins., broke under a centre load, giving for modulus of rupture 277 lbs. per square inch. The same mixture under a direct pull, after 24 hours in air and 55 days in water, gave a tensile strength of 240 lbs.

Relation between Tensile Strength and Modulus of Rupture.—Experiments to determine the relation between tensile strength and modulus of rupture of briquettes and beams of concrete, using the same cement and aggregates for like sets, were made. The briquettes were broken by direct tension, and the beams by cross-breaking. It was found that the modulus of rupture was about one and one-half times greater than the tensile strength. This is about the same relation as that existing in cast iron and stone. The fracture in the beams often passed directly through the stones. The conclusion from this is that the mortar was as strong as the stone or stronger, and consequently the strength of the stone is a factor of the strength of the beams.

TABLE SHOWING RESULTS OF TESTS OF NEAT PORTLAND-CEMENT BEAMS AND BRIQUETTES. BRIQUETTES IN TENSION, BEAMS IN CROSS-BREAKING; BEAMS OF SQUARE SECTION, 1×1×8 INCHES. AVERAGES OF A NUMBER OF EXPERIMENTS.

	Age in Days.	Pounds per Square Inch.	Factor.
Briquettes	30	809	
Beams	30	1242	1.53
Briquettes	112	932	
Beams	112	1406	1.50
1 Portland Cement, 2½ Sand—Mortar.			
Briquettes	30	376	
Beams	30	540	1.43
Briquettes	60	482	
Beams	60	634	1.32
Briquettes	112	493	
Beams	112	679	1.37
1 Portland Cement, 3 Sand—Mortar.			
Briquettes	28	282	
Beams	28	417	1.47
Briquettes	56	328	
Beams	56	512	1.56
1 Portland Cement, 2 Sand, 5 Cinder—Concrete.			
Briquettes	30	110	
Beams	30	183	1.66
1 Portland Cement, 3 Sand, 5 Stone—Concrete. Beams 3×3×30 Inches.			
Briquettes	28	187	
Beams	28	304	1.63

A NEW DRY DOCK AT BALTIMORE, MARYLAND.

THE principal dimensions of this dock are as follows: Length over all, 628 ft.; width on floor, 62 ft.; width on top of keel-blocks, 69 ft.; entrance at top, 80 ft.; entrance at bottom, 60 ft.; width on top of dock, 125 ft.; depth of water on sill at low water,

$22\frac{1}{2}$ ft.; depth at high water, 25 ft. The dock is built of piling and timber construction in the basin portion, with an entrance and power-house combined of concrete and masonry faced with Port Deposit granite in the entrance proper. It is capable of docking a vessel 600 ft. in length and 70 ft. beam, with a draft of $22\frac{1}{2}$ ft. The entrance is 208 ft. out from the shore line; and an excavation was required 164 ft. long, 44 ft. wide, and 36 ft. below low water. The foundation-bed is a hard, compact white clay. A cut-off 3 ft. wide and 3 ft. deep was excavated clear across the entrance and its abutment, and subsequently filled with concrete to prevent leakage between concrete and clay. To protect the projecting end heavy bulkheads were built on either side, each consisting of 10×12-in. tongued and grooved sheet-piling, from 34 to 48 ft. long, driven in two rows 16 ft. apart and braced by guide-piles, 12×12-in. timbers, and $1\frac{3}{4}$-in. iron rods spaced 10 ft. apart. The inner faces of these bulkheads were from 132 to 145 ft. apart. On the south side of the dock a third row of sheet-piling was driven, and between the outer two rows the concrete of the power-house was laid.

The entire area of the dock-basin was excavated by dredging to the depth required, and a coffer-dam was built to close the entrance opening between the bulkheads. As the water was pumped out, braces resting on temporary piles were placed between the bulkheads.

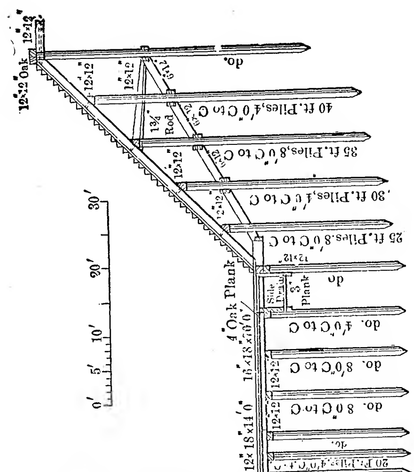
Two rows of 10×12-in. tongued and grooved sheet-piling were driven at each end of the excavations for the entrance masonry.

All the concrete work is made of 1 part "Dragon" American Portland cement, 2 parts sand, and 5 parts broken stone.

In Fig. 67 are shown half-sectionss at entrance and near centre of dock.

The bottom and slopes of the basin are supported by piles, and capped by 12×12-in. timbers.

The floor cross-timbers are made of Oregon pine, 16×18 ins., and spaced 4 ft. apart. Every alternate timber is 70 ft. long, the others are 14 ft. long. All these timbers are drift-bolted to the piles and to each other. The 12×14-in. slope-timbers are framed into the top of the long floor-timbers, and between each



Half Midship Section of Dock-basin.

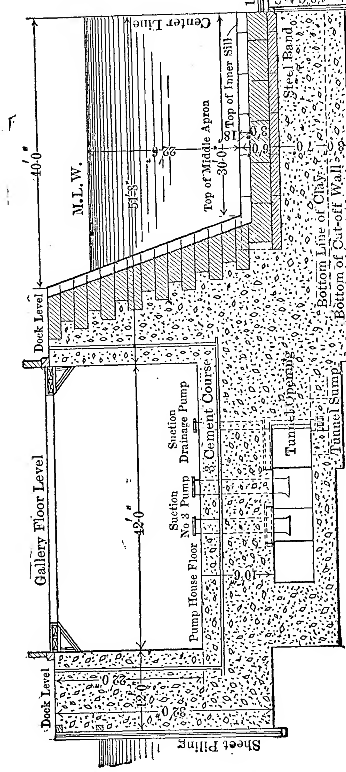


Fig. 67.—Half-section at Entrance of Dock showing Power-house and Tunnel.

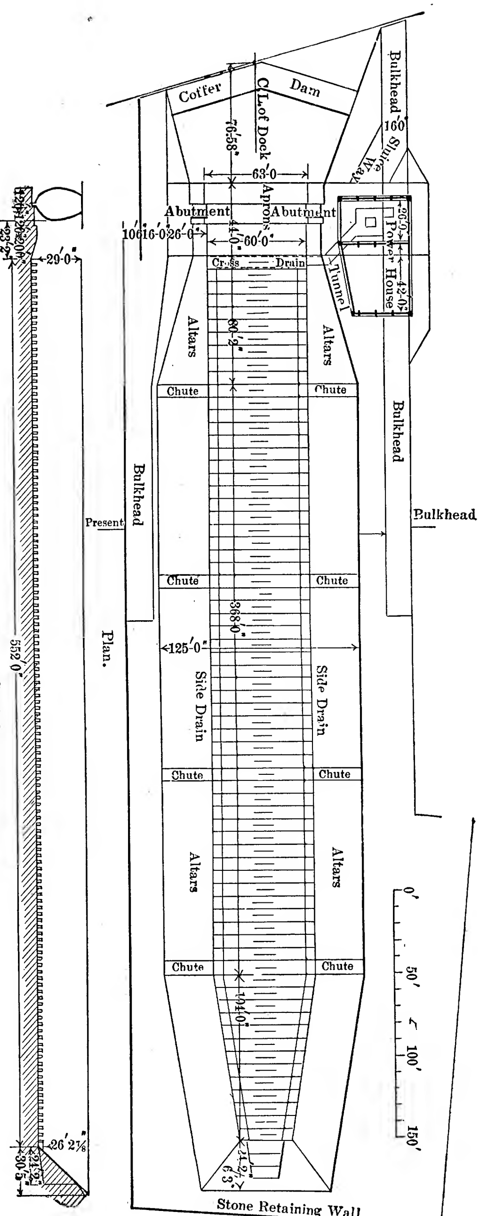


Fig. 68.—General Plan and Longitudinal Section of New Dry Dock at Baltimore, Md.
Built for the Wm. Skinner & Sons Shipbuilding and Dry Dock Co., Baltimore, Md. Ritchie & Ruple, Cleveland, O., Designing and Superintending Engineers. Delaware Construction Co., Wilmington, Del., Contractors.

main slope-timber is an intermediate 8×14-in. slope-timber framed into a filler, which is also framed into the sides of the long floor-timbers. The floor is made of 4-in. oak plank, spiked to joists bolted to the longitudinal capping on the piles. The surface of this floor slopes from the centre to each side and meets the side drains, which empty into a main cross-drain leading to the tunnel under the pumps. The five rows of piles supporting the slopes are braced by 12×12-in. timbers and waling-pieces. The altar-timbers or steps have a rise of 10 ins. and are all securely spiked to the slope-timbers. The top of the slope is finished by

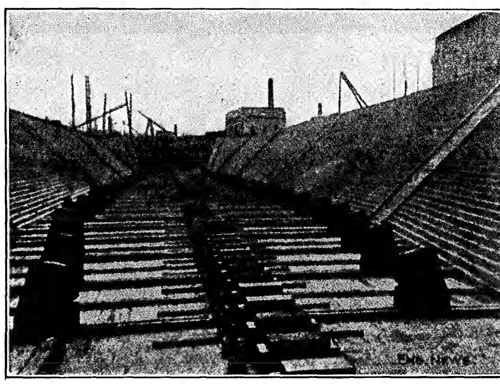


FIG. 69.—GENERAL VIEW OF INTERIOR OF COMPLETED DOCK; LOOKING TOWARDS ENTRANCE.

two 12×12-in. timbers bolted together and drift-bolted to the upper cap. The entrance-gate is a steel caisson set in grooves of the granite facing of the entrance and giving an opening of 57 ft. 8 ins. at low-water line, with $22\frac{1}{2}$ ft. of water on the sill.

The entrance proper and floor of the aprons are faced with Port Deposit granite, finely cut. The sill-stones are 6 ft. thick and weigh from 14 to 17 tons each, and under these is concrete 8 to 10 ft. in thickness. The apron-floor is from 2 to 3 ft. thick. The granite courses in the abutments are 2 ft. high, the stretchers not less than 6×4 ft. in plan, and the headers 3×6 ft. All these stones are doweled together on the beds, and the outer sills are tied to the inner ones by steel bands imbedded in the concrete under them.

For details of power-plant, pumps, valves, drains, tunnels, etc., see *Eng. News*, Jan. 9, 1902.

Fig. 68 shows the general plan, and Fig. 69 a general view.

PROPORTIONS FOR CONCRETE IN RESERVOIR CONSTRUCTION, ETC.

At Canton, Illinois, the proportions for concrete in reservoir construction were fixed at 1 cu. ft. Portland cement, $3\frac{1}{2}$ cu. ft. sand, and $8\frac{1}{3}$ cu. ft. of crushed stone, assuming 40 p. c. of voids in broken stone and 30 p. c. in sand.

For concrete in arches of the New York Rapid Transit Railway the required proportions were: 1 part Portland cement, 2 parts sand, and 4 parts broken stone, corresponding to 45 p. c. voids in stone and 50 p. c. in sand.

CONCRETE ROAD-BED CONSTRUCTION FOR STREET RAILWAYS.

In the following drawings the details of some typical road-bed construction for street railways are given.

Fig. 70 shows the construction in Kansas City. The concrete

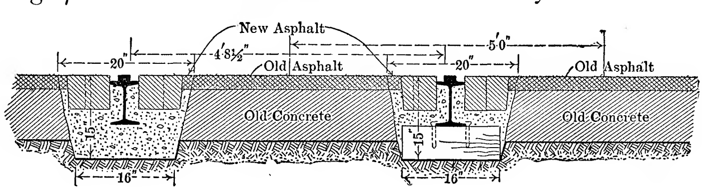


Fig. 70.—Street-railway Track with Concrete Stringers; Metropolitan Ry.; Kansas City, Mo. (E. Butts, Chief Engineer.)

is composed of 1 part Portland cement (by measure), 1 part natural cement, 4 parts sand, and 10 parts crushed stone. As shown, under the rail on the right-hand side a wooden block is used to which the rail is spiked; such blocks are only temporary and are removed when the concrete is placed. The construction on the left half is the permanent one. The concrete is 6 ins. thick below base of rail. The trench under each rail is 20 ins. wide t top and 16 ins. at bottom.

Fig. 71 shows concrete construction, Toronto, Canada. Tie-bars $\frac{3}{8}\times 2$ ins. are set 6 ft. apart and held by nuts to the web of the rails, which are of the grooved girder kind, $6\frac{1}{2}$ ins. high.

Two types of construction employed in Buffalo, N. Y., are

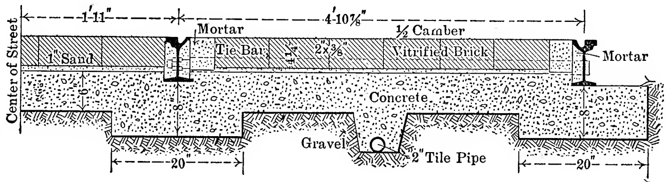


Fig. 71.—Street-railway Track with Concrete Floor; Toronto Ry. Co. Toronto, Ont., Canada.

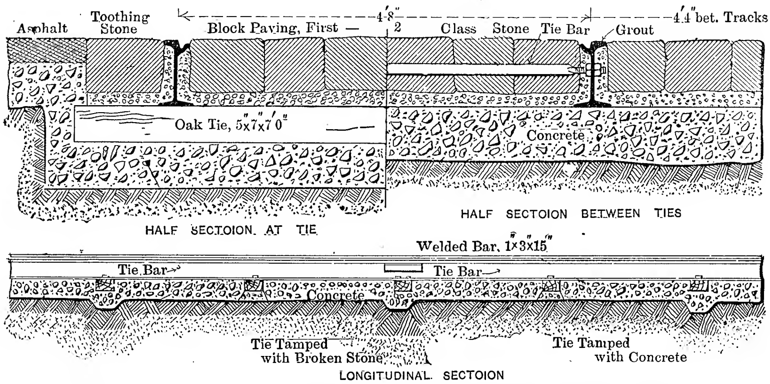


Fig. 72.—Street-railway Track with Concrete Floor; Buffalo Ry., Buffalo, N. Y. (C. C. Lewis, Engineer of Ways and Buildings.)

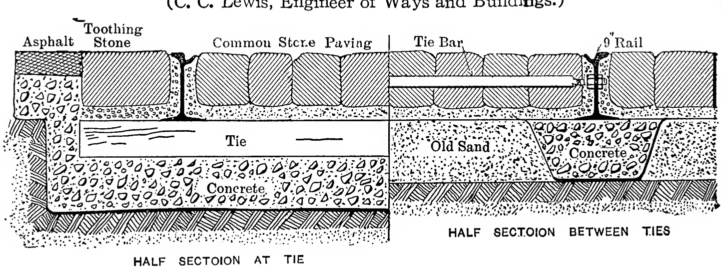


Fig. 73.—Street-railway Track with Concrete Stringers; Buffalo, N. Y.

shown in Figs. 72 and 73. The rails are 9-in. girder, 60 ft. long, drilled for tie-bolts at ends and at intervals of 10 ft. The rails are laid on oak ties 5×7 ins., 7 ft. long, spaced 5 ft. Alternate ties are tamped with stone ballast, and the other ties embedded in concrete, and a concrete stringer 8 ins. deep and 15 ins. wide is constructed under each rail.

All the concrete is composed of 1 part Portland cement, 3 parts sand, and 5 parts broken stone. The 9-in. 90-lb. rail or $6\frac{1}{4}$-in. 60-lb. rail is used.

In Cincinnati, O., the Lorain Steel Co.'s rail weighing 109 lbs. per yard is used; it is known as the Trilby rail. In Fig. 74

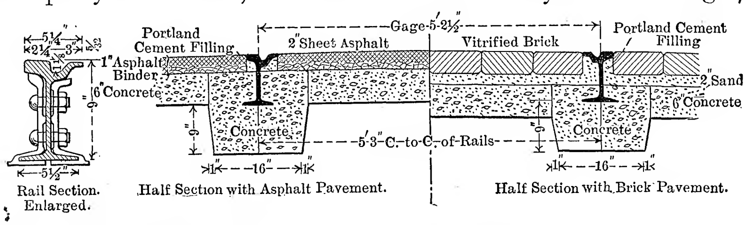


FIG. 74.—STREET-RAILWAY TRACK WITH CONCRETE BEAMS AND GROOVED GIRDER RAILS; CINCINNATI, OHIO.

are shown the details of construction. The rail is 9 ins. high, 5 ins. wide at base and $5\frac{1}{4}$ ins. at top, with a flaring groove $1\frac{1}{8}$ ins. deep. The guard side of the groove is $\frac{1}{16}$ in. lower than the running side on tread. The splice-bars are 26 ins. long, with eight 1-in. bolts in two rows, the bolts being staggered. The rails are spiked to oak cross-ties at intervals of 10 ft., and are connected by tie-bars $\frac{5}{16}\times1\frac{1}{2}$ ins., 5 ft. apart. The rails are embedded in concrete stringers, 9 ins. deep below the base of rail, and extending 3 to 6 ins. above the base, with a width of 18 ins. at top and 16 ins. at bottom. The concrete is made of Portland cement and limestone. The street has a 6-in. concrete foundation made with natural cement, on top of which is a 2-in. cushion of sand for vitrified-brick paving, or with a 1-in. asphalt binder course for a 2-in. surface of sheet asphalt. (See *Eng. News*, Mar. 6, 1902.)

A CONCRETE-LINED RESERVOIR.

AT bottom of reservoir the concrete is 8 ins. on the sides, 15 ins. at top, 18 ins. in thickness. The proportions used were: 1 part native Portland cement, 2 parts clean and sharp sand, and 3 parts crushed stone. The concrete to be spread in layers not over 6 ins. in thickness and rammed until free mortar appears on the surface. Sufficient water to be used in mixing to permit of ramming. The entire surface to be washed with a mortar of 1 part cement and $1\frac{1}{2}$ parts of sand, made of such consistency with water that it may be spread with a wooden trowel and then washed with a coat of cement-wash; the concrete surface also to be washed with cement. The crushed stone to have no pieces over 1 inch diameter or less than $\frac{1}{8}$ inch, all dust being removed. This follows the antiquated idea of screening out the fine particles of stone. Other requirements of the specifications were that neat-cement cakes $\frac{1}{2}$ inch in thickness, when set hard, shall be immersed in water for three days, and must show no cracking or distortion. The same subjected to a steam-bath and afterwards immersed in water at a temperature of 180° F. shall show no signs of distortion or disintegration. At least 85 p. c. shall pass a sieve, No. 100, containing 10,000 meshes per square inch. Neat-cement briquettes one hour in air, 23 hours in water, shall show a tensile strength of 150 lbs. per square inch; one day in air and 6 days in water, 400 lbs.

PLANS FOR THE PROTECTION OF THE CITY OF GALVESTON, TEXAS, FROM FLOODS.

In the year 1900 the city of Galveston was flooded and to a great extent destroyed. A board of engineers, with the view of preventing another such catastrophe, recommended the construction of a solid concrete wall, about three miles in length; the top of this wall to be 17 ft. above mean low-water, or 1.3 ft. higher than the highest point reached in the storm of 1900; and to raise the street grades by filling in behind this wall.

In Fig. 75 is shown a cross-section of this sea-wall. The sea-face of the wall is curved so that its upper portion will be nearly vertical, with the purpose of giving the wave-motion an upward direction before reaching the top of the wall. The wall is to be built on piles and protected from undermining by sheet-piles and riprap. Its width at bottom, 1 ft. above mean low water, is 16 ft.; and at top, 17 ft. above mean low water, it is 5 ft. The embankment, for a width of 35 ft. from the wall, is to be paved with vitrified brick set on edge, providing a driveway of 30 ft. and a sidewalk 5 ft. in width. The west end of the city is to be protected by a levee 300 ft. in width, with slopes 1 in 25.

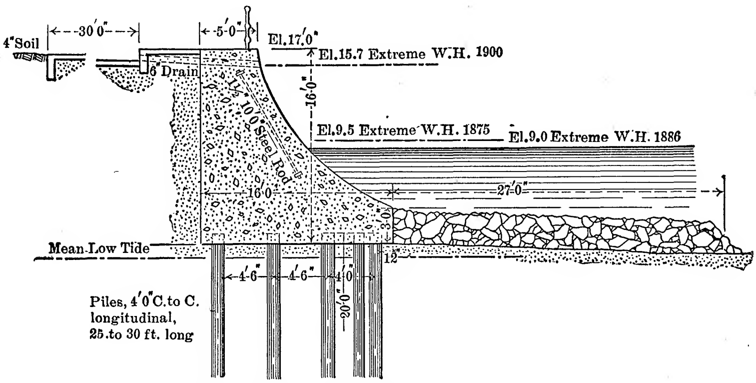


FIG. 75.—CROSS-SECTION OF PROPOSED SEA-WALL ON GULF SIDE OF GALVESTON.

The estimated unit cost of the work is as follows:

Excavation for sea-wall, per linear foot	$0.50
Piles driven, per linear foot	.10
Sheet-piles driven, per linear foot	.25
Concrete, per cubic yard	8.00
Steel rods and cast-iron washers, per pound	.03
Hand-rail, per linear foot of wall	1.50
Riprap, in place, per ton	2.00
Filling, per cubic yard	.15
Brick paving, per square yard	1.25
Brick curb, per linear foot..................................	.90
Soil and Bermuda grass, 60 ft. wide, per linear foot	1.00

The filling in these estimates includes the embankment behind the sea-wall and the levee on the western limit. At the unit prices above named the cost will be as follows:

Breakwater or sea-wall, 17,700 linear ft. @ $66.50 ...	$1,177,050
Add 10 p. c. for engineering and contingencies........	117,705
Filling..	2,210,285
Total cost..	$3,505,040

(See *Eng. News*, April 24, 1902.)

SHAFT-SINKING BY THE FREEZING PROCESS IN GERMANY.

IN order to reach a bed of salts of potash at a depth of 459 ft. below the surface, the attempt was made to reach this depth by means of an ordinary masonry-lined shaft. To the depth of 407 ft. the material is a badly fissured and irregular gypsum. Overlying the potash formation is a 50-ft. stratum of hard and compact gypsum. The first 46 ft. of shaft was 26.56 ft. in diameter, the lower portion was 19.68 ft. in diameter. The upper and larger section of shaft was lined with masonry 2.62 ft. in thickness. Inside of this an annular cast-iron ring, 19.68 ft. in diameter, was forced down by the action of powerful hydraulic presses. The water encountered to a depth of 23 ft. was lifted by eight pulsometer pumps having an aggregate capacity of 1553 cu. ft. per minute.

Both the masonry and the iron lining were forced out of line, which at a depth of 105 ft. amounted to 4 inches. All attempts to straighten the lining failed. The inflowing volume of water increased to 1412 cu. ft. per minute, and finally the earthy material was forced in under the bottom and filled the shaft to a height of 26 ft. It was then determined to resort to the freezing process. Owing to the saline character of the water, which contained 3 p. c. salt at a depth of 105 ft., experiments were made on different saline solutions containing 4, 8, 10, and 12 p. c. of NaCl. These solutions were subjected for a period of 24 hours to a temperature of $-12°$ C.; the 4 p. c. solution was solidly frozen, the others to a less degree, and the 12 p. c. solution was little affected at the end of 48 hours' exposure to the freezing process. This being conclusive, thirty pipes were sunk on a circumference of 29.5 ft. in diameter and to a depth of 413 ft. The freezing machinery of the Fixary system, having double the capacity required in freezing non-saline soil, was installed. The central tubes were then sunk and the sets of tubes connected by a series of circulating pipes. The freezing operation was then commenced with a temperature at the bottom of the shaft of $-4°$ C.

The main portion of the enclosed core was removed by explosives; the portion near the walls of the shaft was removed by hand. The old cast-iron lining was removed by means of warm air-currents thawing the material in contact with the iron lining; a segment was broken from the ring and whole iron lining removed. The outside diameter of the masonry lining is 26.5 ft., and the inside diameter 18 ft. (See *Eng. News*, April 24, 1902.)

SINKING A SHAFT IN QUICKSAND BY THE FREEZING PROCESS.

THE sinking of a shaft in quicksand is described in *The Iron and Coal Trade Review*, June 5, 1903. The shaft is at the Laura and Vereeninging Colliery, in the Limburg province, Holland. It is 270 ft. deep to the coal measures, and 15 ft. inside diameter.

"At first a temporary shaft, with a diameter of about 30 ft., was sunk to a depth of 28 ft., and was provided with a provisional timbering consisting of planks and rings of channel-iron. From the bottom of this shaft 24 bore-holes were made, intended for the reception of the freezing-tubes. These holes were sunk on the periphery of a circle with a diameter of 24 ft., so that the freezing-tubes were at a distance of about 36 ins. from each other. They are of a diameter of 4 or 4½ ins., and accommodate the small descending pipes, which only measure 1¼ ins. across. The latter communicate by means of valves with a distributing-ring, and the former with a return-ring introduced in order to be able to regulate the flow of the freezing-solution. In the middle of each freezing-tube there is inserted an elastic union which admits of its expansion and contraction with the fluctuations of the temperature. The freezing-liquor, which consists of a solution of chloride of magnesium, leaves the refrigerators at a temperature of 20° C., and passes into the distributing-ring, then down the small descending pipe as far as the bottom of the freezing-tube, in order to ascend up the annular space between the pipe and the tube, till it reaches the return-ring, from which it is conveyed by a pipe back again to the refrigerators, where the temperature is once more lowered. By means of the various valves any one of the freezing-tubes can be cut out of the system. This operation is rendered necessary when one of them becomes stopped up or leaks.

"For the production of the cold there was a double compressor of which the piston diameters were 5 ins. and $7\frac{1}{2}$ ins., and the stroke 18 ins. This machine was driven by a steam-engine through the medium of a belt. The carbonic acid was drawn in at a pressure of 220 lbs. per square inch, and compressed to from 1000 to 1200 lbs. per square inch. The compressed and consequently heated gas was next sent through a cooler, where it lost some of its heat, and then was passed into the two condensers in which it was liquefied, and finally the carbonic acid was conveyed through a third condenser to the refrigerators. Before entering the worms of this apparatus it passed through an expansion-valve, which suddenly brought the pressure from 1100 lbs. down to about 200 lbs., causing a most intense absorption of heat, or, in other words, the production of intense cold. The refrigerator is a cylindrical vessel with a diameter of 56 ins. and a height of 110 ins., which is filled with the freezing-mixture, consisting of a solution of magnesium chloride 30% strong. The spiral worm-tube with the carbonic acid winds inside this vessel and causes the temperature of the solution to sink to 20° C., at which it is pumped into the distributing-ring and from thence flows down the freezing-tube."

The bore-holes were carefully tested to see that they were perfectly vertical, for any serious deviation from the vertical would leave an unfrozen place in the wall of ice. Rotary borers were used, and in three months' time the 24 holes were down, due to irregular working of the plant. It took eight weeks to freeze a wall of ice 20 ins. thick, and at the end of twelve weeks the ice-wall was deemed thick enough to begin excavation. The frozen material was excavated with chisels and hammers, a soft unfrozen core considerably facilitating the work. This sinking (242 ft.) consumed 75 working days, or at the rate of 68 ins. per working day. In widening out the shaft in the coal-bearing strata a collapse occurred on the south side, filling the shaft with 40 ft. of sand and water. Water was then pumped into the shaft, filling it completely. Twelve more bore-holes were driven in the shaft close to the sides, and six more holes on the south side of the shaft and 18 ft. from its centre. This area was frozen and the work of excavation again begun, and $8\frac{1}{2}$ months after the accident had occurred the bottom of the shaft was again reached.

TESTS OF STRENGTH OF MORTAR WITH SAND AND LIMESTONE SCREENINGS.

In these tests Dyckerhoff German Portland cement, Potomac River sand, and Maryland limestone were used. The briquettes were in air one day and in water six days.

Proportions by Weight of			
Cement.	Limestone Screenings.	Potomac River Sand.	Strength in Lbs. per Sq. In.
1	0	0	493
1	1	0	291
1	0	1	217
1		0	208
1	0	2	129
1	3	0	157
1	0	3	108
1	1	1	209

These tests show that limestone screenings used instead of sand in the same proportions produce a stronger mortar than is obtained by the use of Potomac River sand. And a fair deduction is that expense is incurred in screening the stone for concrete to obtain a mixture having less strength. (See *Eng. News*, April 24, 1902.)

The author's experience fully confirms the above conclusion. He always succeeded in obtaining very much better concrete when using unscreened stone than when using the screened. The impalpable dust was generally blown away, a very gentle breeze being sufficient to effect this purpose.

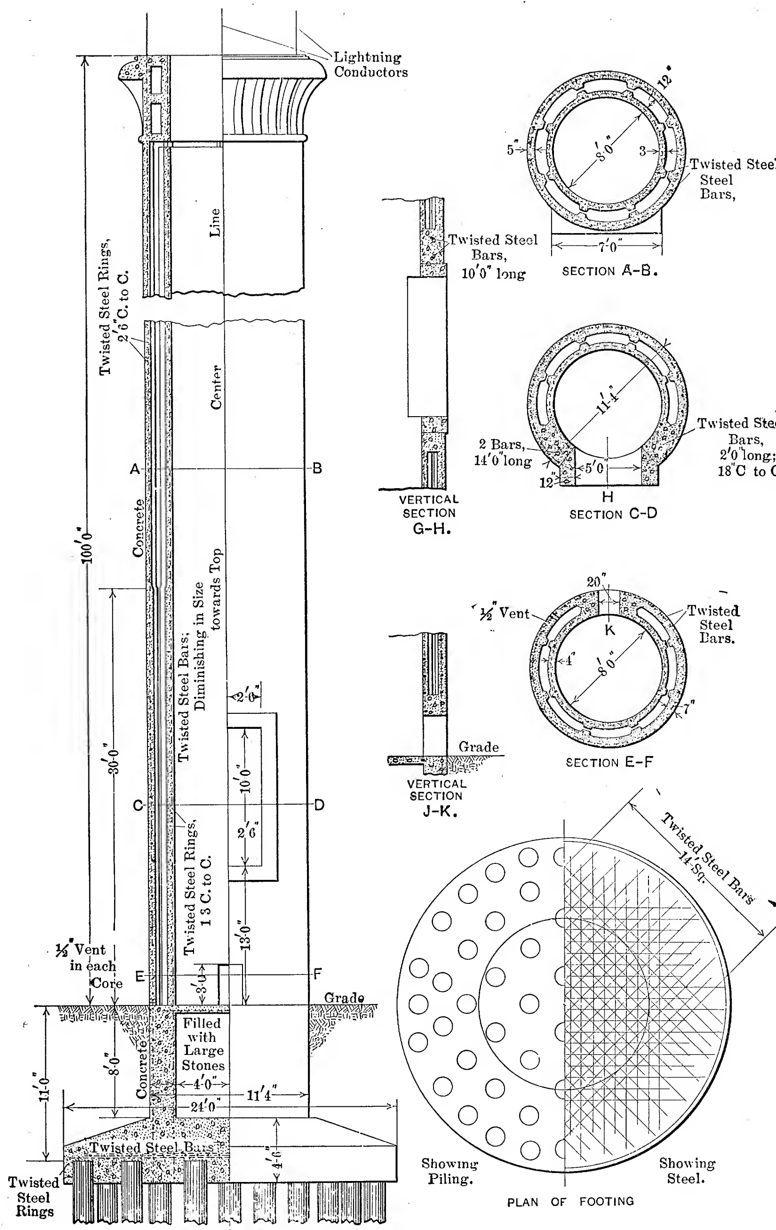


FIG. 76.—REINFORCED-CONCRETE CHIMNEY (RANSOME SYSTEM).

FOUNDATION FOR HIGH CHIMNEY.

A CHIMNEY has been constructed in Jersey City of concrete reinforced with cold twisted steel rods, following the Ransome system. The stack is 108 ft. high above the top of foundation, cylindrical in form, outer and inner diameters 11 ft. 4 ins. and 8 ft. respectively. The shell is double, with annular air-space. At the top an ornamental cap of concrete strengthened with steel is placed. At 8 ft. from bottom it has a gradual spread extending to bottom, where it rests directly on the foundation of piles. The concrete is made with crushed trap-rock, the fine particles of which are used in the place of sand. No other sand was added. The bottom of the flue is at a point 8 ft. above the bottom of the stack; above this point the walls are double. The inner shell or flue-lining is 4 ins. thick; the outer shell varies in thickness from 7 ins. at bottom to 4 ins. at top, by means of offsets on its inner face. Both shells are reinforced by vertical and circumferential twisted steel rods. Below the point 8 ft. from the bottom the walls were solid and 20 ins. in thickness, the hollow interior filled with stone, topped off with a concrete floor, forming the bottom of the flue. The site of the chimney is a soft marshy material extending to a depth of 50 ft. below the surface. Piles, 56 in number, were driven through this soft material and 5 ft. into the underlying hard-pan. Owing to the character of the material, which gives little lateral support to the piles, the load on each pile was limited to $7\frac{1}{2}$ tons, being considered a safe load for a long column. Concrete was placed around the heads of the piles to the depth of 18 inches. And around the outer piles two circumferential rods were embedded in the concrete to prevent possible spreading of the piles or cracking of the concrete. Upon the piles was placed a concrete layer 3 ft. in thickness, with four layers of steel reinforcing-rods crossing each other. The concrete was made quite wet and well rammed when placed.

Four of the vertical reinforcing-rods were well grounded, and their upper extremities which reached several feet above the top of the stack were tipped with copper, in order to act as lightning-conductors.

There are 1460 cu. ft. of concrete in the foundation and 3514 cu. ft. in the stack proper. The weight of the twisted steel rods is 8020 lbs. Allowing 144 lbs. per cubic foot for the weight of the concrete, the total weight of the chimney is 362 tons, about $6\frac{1}{2}$ tons to each pile. Assuming a wind pressure of 25 lbs. per square foot of vertical projection, the maximum unit stress in the concrete is 350 lbs. per square inch compression. The cost was $3500, or about $17.50 per cubic yard of actual volume. For the details of construction see *Eng. News*, Dec. 19, 1901. The chimney was built by the Morris Building Co., and designed by the Ransome Concrete Co.

Drawings showing section of stack and plan of foundation are shown in Fig. 76.

SIMPLE AND REINFORCED CONCRETE.

Reinforced Concrete.—The present practice of reinforcing concrete beams with straight or twisted rods, wire netting, and angle-iron has become so common that more than a passing notice seems desirable or necessary. It is not always apparent whether the purpose of the steel or iron is to strengthen the concrete or whether the concrete is intended to strengthen and preserve the steel or iron. One thing seems evident, in many structures at least: that the concrete as used will not carry the loads without the iron, while in many such structures the iron will carry the load without the aid of the concrete. In such cases the concrete only acts as a filler and as means of protecting the iron from rusting. Others claim that each material bears a portion of the load, and that in proportion to their respective carrying capacity. How far such a contention may be relied upon must be at least problematical. As all such structures can be built of concrete alone, or of iron and steel alone, the problem seems to resolve itself into one of immediate and ultimate economy; though engineers, like other people, are liable to do things without being able to assign any special reason, and take up new suggestions and do things simply because other engineers have done the same things under similar circumstances with structures that afterwards proved satisfactory.

On the Illinois Central R.R. there is a concrete bridge 77 ft. long, consisting of four arch-spans. The spans are 15 ft. in the clear, with a radius of 20 ft. for the crown and 2 ft. for the haunches. The concrete is 18 ins. thick at the crown, and over each arch are laid 9-in. I beams 17½ ft. in length. There are seven of these beams under each track, spaced 18 ins. centre. At the centre of the arch there are 3 ins. of concrete below and 6 ins. above the beams. The concrete for the bottom 3 ins. and for 1½ ins. above the beams is made of small stone. The stone ballast 18 ins. in thickness is laid on the concrete; in this the ties are embedded. The piers are 3 ft. thick, and the abutments 4 ft. at the springing-line. The bridge contains 725 cu. yds. of concrete. The cost of this structure is said to be less than that of a single-track steel structure of modern design and weight.

The beams considered alone will safely carry not less than 90,000 lbs. The concrete, either directly or indirectly, by giving lateral support to the beams, is supposed to carry any load in excess of 90,000 lbs. that may come on the arch. (See *Eng. News*, July 18, 1901. In the same issue are found the specifications for concrete on the Illinois Central. The following are a few notes taken from these specifications:)

Crushed Limestone.—This shall be made by crushing tough, hard, clean limestone, and screening same through 2-in. meshes or holes. The engineer or inspector in charge shall reject crushed limestone which may have any of the following defects:

"A," containing more than 1 p. c. of the earthy or clayey matter; "B," more than 20 p. c. of fine stone or stone dust, less han ½ in. in size; "C," more than 5 p. c. of soft or rotten stone, which can be crushed or powdered up in the fingers; "D," more than 10 p. c. of flat stone larger than 2 ins. in greatest dimension; "E," more than 15 p. c. of crushed stone larger than specified (passing through a 2-in. mesh), unless there be an equal amount of fine material less than ½ in. in size. The author believes the unscreened stone, the-run-of-the-crusher, will give better results. But admitting the wisdom of such requirements, it is impracticable to determine the specified percentages or to enforce such requirements, and with them the cost must be materially greater.

Sand.—The usual requirement is specified, clean sharp sand, pit sand preferred—a sand which will not pass through a sieve having 30 meshes to the inch.

Cement.—No tests are prescribed, but it is stipulated that the brand of cement must be accepted by the inspector, and, where practicable, samples are to be subjected to the usual one-day and seven-day tests. The inspector shall, from time to time, make pats of neat cement and cement and sand, in order to satisfy himself that a cement of uniform character is delivered. This specification is unusually liberal, but is about all that can be required where large quantities of cement are used, and is probably safe enough where nothing but well-known and standard brands are used.

Natural-cement Concrete.—This may be used in works totally submerged below low-water mark, or not liable at any time to be exposed to the weather. In all cases where foundations are liable to be exposed to the action of water, or where the material in the bottom of the excavation is soft or of unequal firmness, Portland-cement concrete must be employed in foundation work. This seems an unnecessary requirement, certainly, except for foundations of the largest and most important structures. The concrete shall be made in the proportions (by measure) of 1 part cement, 2 parts sand, and 5 parts of crushed stone. For the Portland-cement–concrete foundations 1 part cement, 3 parts sand, and 6 parts crushed stone may be used; or, if required by the engineer, a stronger concrete, made, for the natural cement, of 1, 2, and 4 parts, and for Portland cement, of 1, 2, and 5 parts.

Portland-cement Concrete.—For concrete for the bodies of piers and abutments, all wing-walls, and the bench-walls of arch culverts, the proportions shall be 1 cement, 2½ sand, and 6 crushed stone, or for special strength the proportions may be 1, 2, and 5, all by measure. For arch rings, parapet head-walls, and coping, Portland-cement concrete in the proportions of 1, 2, and 5 shall generally be used. Bridge-seats of piers and abutments and of copings to carry pedestals for girders or longer spans shall generally be made of crushed granite and Portland cement in the proportions (by measure) 1 part cement, 2 parts fine granite screenings, and 3 parts granite screenings, the larger of which shall not exceed $\frac{3}{4}$ in. in greatest dimension.

Mixing.—The specifications require the mixing to be done in a manner substantially the same as that previously described in this volume (see pages 9 to 20).

It is generally specified that no concrete is to be placed under water: the water must be pumped out. Over and around piles where the bottom of the excavation is soft, broken stone may be spread and rammed into the earth before placing the concrete. In no case shall a dry mixture of cement, sand, and stone be used in making the foundation. It may be mixed with a less proportion of water, but must be well mixed before placing. In case of a bed of sand and gravel a cement grout, 1 cement and 2 sand, may be forced to the depth of a foot or more, by means of pipes, into the gravel and sand. This, allowed to stand for 24 hours or more, will generally result in a concrete sufficiently compact to permit of pumping out the water. In long walls and arches provision is made for expansion by means of expansion-joints. The arch ring may be built in separate sections of about 25 ft. in length, simply abutting against each other.

Concrete-steel Column Footing.—In Fig. 77 is shown a concrete-steel column footing designed by the St. Louis Expanded Metal Company. The footing consists of a flat block of concrete having embedded in its lower half a grillage of corrugated steel bars of the form shown in Fig. 77, *a*. The bars vary in size from $\frac{1}{4}$ inch to 2 inches square. The footing as shown costs $51.51, whereas a similar plain concrete footing will cost $61.25. The cost, however, is not the only consideration. The ordinary I-beam concrete has been found by experiment to lose in time the adhesion between concrete and iron, and these cannot be depended upon to act together either in foundation or arch construction. The plan proposed is to substitute a number of small corrugated steel bars. It is desirable to have the faces of the corrugations at right angles to the axis of the bar to insure a mechanical form of bond which cannot be destroyed. Such a bar cannot be rolled, and cast-iron bars are deficient in strength. The angle of friction between steel and concrete is about 24°. If the faces of the corrugations do not depart from the normal to the axis by a greater angle, the bond will be permanent, the two materials will act together and the bars will not slip in the concrete, and the concrete is prevented from cracking.

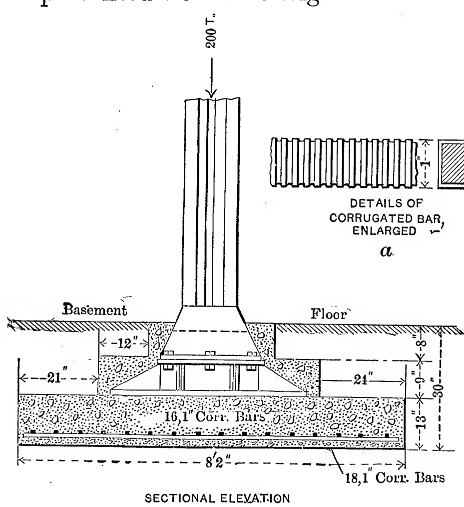


FIG. 77.—STEEL-CONCRETE COLUMN FOOTING WITH ROLLED CORRUGATED BARS. (St. Louis Expanded Metal Co., St. Louis, Mo., Builders.)

Flexibility.—With the reinforced concrete, where the bars are properly distributed throughout the mass, elongation twenty times that capable of occurring in plain concrete may develop without cracking and without lessening the full tensile strength of the concrete. Considerable deflection of floors and slabs of concrete will not cause cracks. Reinforced concrete also prevents distortions resulting from temperature stresses and shrinkage from producing injurious effects. (See *Eng. News*, April 3, 1902.)

The above statements are based upon conclusions reached by M. Considère. Later experiments throw at least much doubt upon the reliability of M. Considère's findings. Engineers are hardly justified in accepting from an insufficient number of experiments such conclusions, especially where such extravagant and seemingly unreasonable and improbable results are reported. Especially is this true where foundations are concerned. If the foundation-bed is not overloaded, no danger need be apprehended.

But if it yields, the concrete base will be sure to give way under heavy loads.

Concrete and I-beam Tunnel Lining.—An interesting example of the concrete-iron construction is found in the lining of the Aspen Tunnel, Wyoming. The material through which the tunnel was driven is a shale of carboniferous formation with an occasional stratum of yellow sandstone, having a dip of from 20° to 30°. The shale approaching a soapstone in character proved to be very treacherous and difficult to hold in place. Much water was also encountered. A very strong timber lining was tried, but, owing to the great pressure, the lining was forced in steadily and gradually. Solid-wall timbers were unable to resist the pressure. Whether this was caused by a swelling of the shale due to oxidation by contact with the air or by direct pressure is not definitely known.

Large wall-plates were compressed to about one-half their original thickness by the great pressure to which they were subjected. It was therefore determined to employ, instead of timber, a combined concrete and steel lining. The steel framework consists of 12-in. I beams weighing 55 lbs. per foot and spaced from 12 to 24 ins. between centres, depending on the pressure. Each beam is made of three pieces or segments bent to the proper curves, the top segment having a radius of 8 ft. 2 ins., and the two side segments having a much larger radius. Each rib weighed 3446 lbs. The centre height is 20 ft. 10 ins. above top of ties; the greatest width is 18 ft. 3 ins. The beams rest on cast-iron shoes. The thickness of concrete is from 4 to $6\frac{3}{4}$ ins. inside of beams, and reaches a thickness of from 2 to 3 feet where the beams are embedded, as indicated in Fig. 78.

The concrete foundation extends across the tunnel and has a depth of $5\frac{1}{2}$ ft. in the centre, and is strengthened by a layer of old rails embedded in it and reaching across the tunnel. The concrete is composed of 1 part cement, 3 parts sand, and 6 parts broken stone. There are about $8\frac{3}{4}$ cu. yds. concrete per linear foot. The clear dimensions of the concrete section are 17 ft. $1\frac{1}{2}$ ins. in greatest width at $7\frac{1}{4}$ ft. above base of rail, and $20\frac{1}{2}$ ft. high above base of rail.

In Fig. 78 are shown details of cross-section, and Fig. 79 shows a view of the beams in place, and also the lower portion of the concrete lining. (See *Eng. News*, March 6, 1902.)

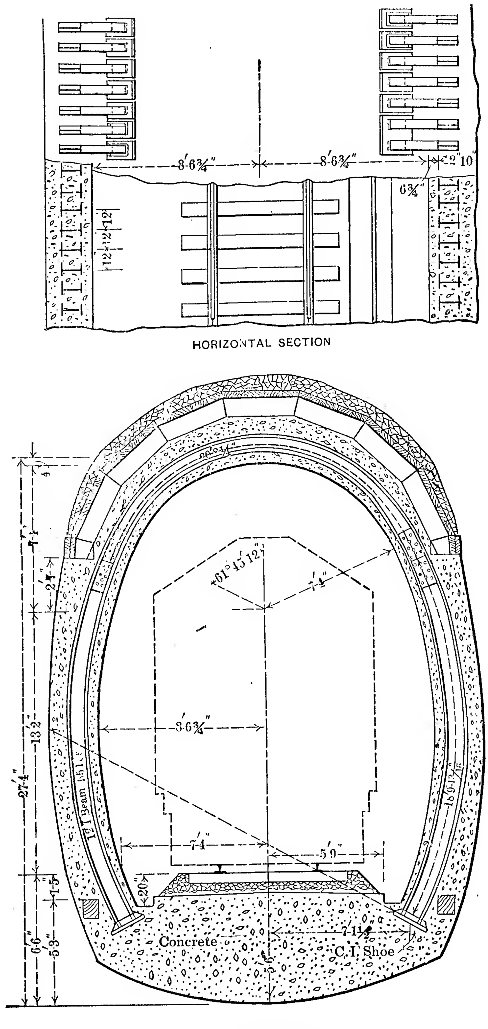


Fig. 78.—Plan and Section of Concrete and I-beam Lining.

FOUR-TRACK, TWO-TRUSS BRIDGE OVER THE CHICAGO DRAINAGE CANAL.

(*Engineering News*, Sept. 12, 1901.)

THE piers for this bridge rest on a hard clay foundation-bed, at a depth of 9 ft. below the bed of the channel. The foundation-bed carries a load of 5000 lbs per square foot.

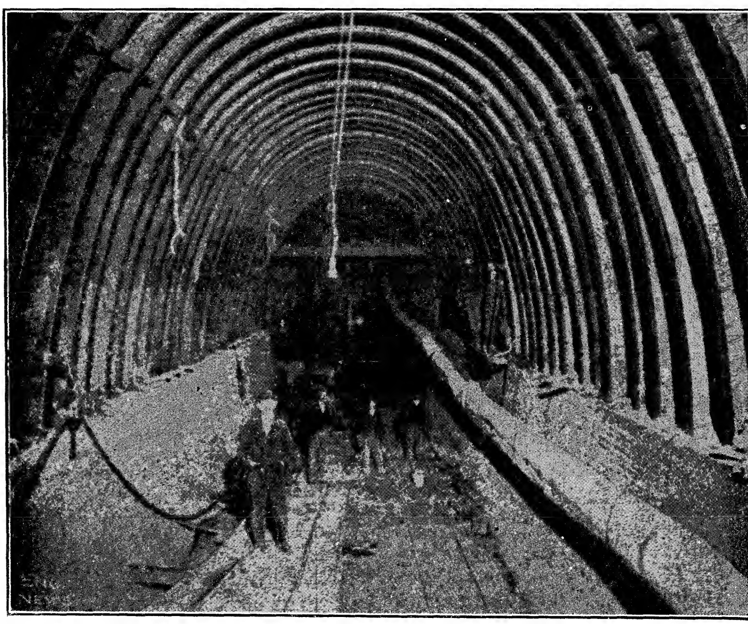


FIG. 79.—VIEW SHOWING I-BEAM ARCH RIBS IN PLACE AND CONCRETE FILLING COMPLETED PART WAY UP ON EACH SIDE.

The pivot-pier is octagonal, 43.75 ft. in diameter. It is built hollow, the shell, 7 ft. in thickness, enclosing a hollow space 28.75 ft. in diameter; this space is filled with earth.

The masonry consists of Portland-cement concrete and Bedford stone masonry built in courses 18 to 24 inches in height.

Two pairs of plate girders cross the hollow space and are supported on the masonry wall. The centre-pin for the turntable is supported by the girders at their intersection. A complete description and stress-sheet of the trusses is found in *Eng. News.*

Fig. 80 shows a general view of the pivot-pier, Fig. 81 a half-section, and Fig. 82 part plan.

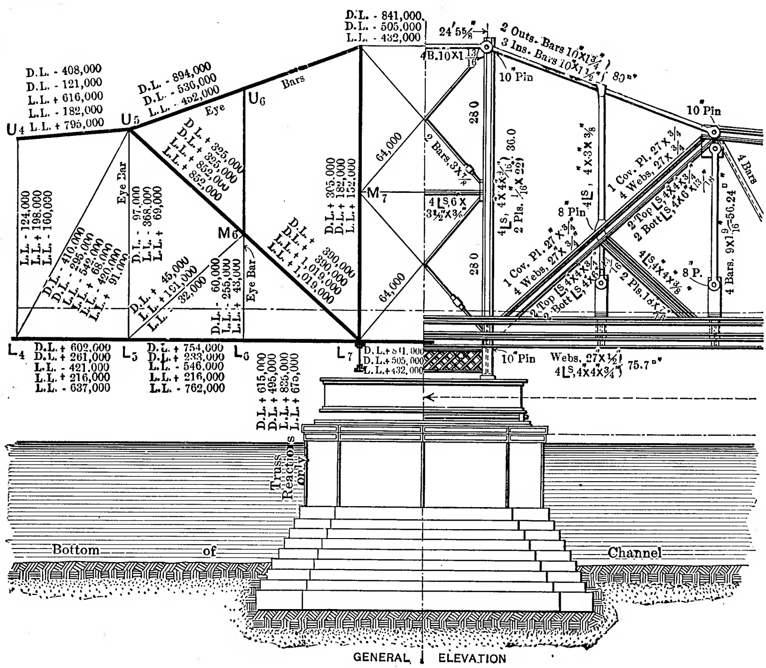


FIG 80.—PIVOT-PIER.

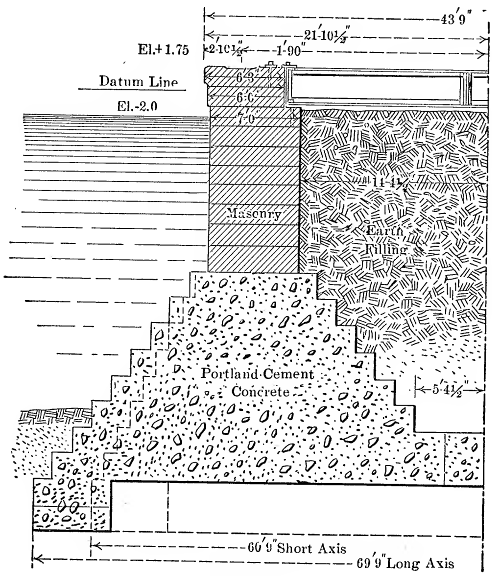
The unit prices for substructure were as follows:

Excavation (average).......	$0.51 per cu. yd.
Portland-cement concrete. ..	7.30 " " "
Stone masonry.............	13.35 " " "

Total for substructure, $51,353.00

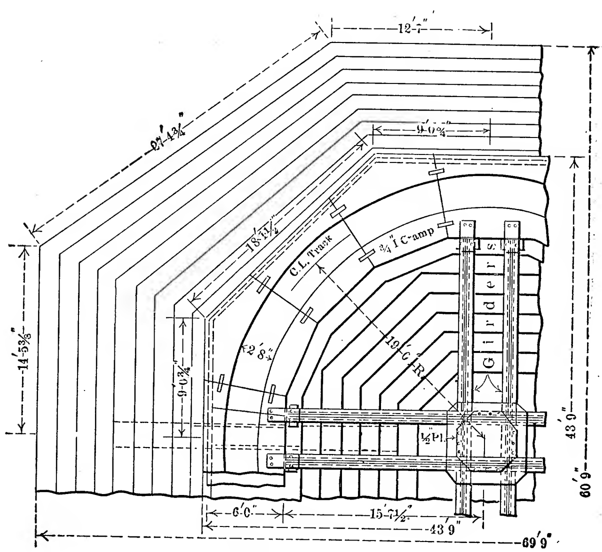
TEST OF A STEEL-CONCRETE SIDEWALK VAULT-LIGHT SLAB.

THE vault-light slab was of the concrete and twisted-steel construction. The test was made under the direction of Mr. Wm. B. Parsons, Chief Engineer, New York Rapid Transit



HALF SECTION

FIG. 81.—CENTRE-PIER DETAILS.



PART PLAN.

FIG. 82.—CENTRE-PIER DETAILS.

Railway, now a member of the Isthmian Canal Commission. The slab tested was built by Tucker & Vinton, of New York City. The special slab was built of two 12-in. I beams, $13\frac{1}{2}$ ft. in length, and spaced 5 ft. between centres, supported at ends on blocks. These beams were connected by three 5-in. I beams, their top being 2 ins. below the top flange of the main beams. This iron frame formed the support of the slab proper. The thickness of the slab was 2 inches, with ribs on the under side acting as fillers and to give the slab a bearing on the 5-in. beams. The special slab tested was constructed by placing a series of bull's-eyes on circular lenses, $2\frac{5}{8}$ ins. in diameter, in rows $3\frac{5}{8}$ ins. centre to centre, and filling the spaces between them with Portland-cement mortar in which was embedded a $\frac{1}{4}$-in. twisted steel bar of square section. The bull's-eyes had a slight taper, with circumferential ribs, and were cupped or concave on the under side.

Fig. 83 shows a transverse section of a standard slab. The method of testing, the loads applied, and the results are quoted from *Eng. News*, Sept. 12, 1901, as follows:

"The method of testing the slab illustrated was to place a steel disk $8\frac{1}{2}$ ins. in diameter on it at the centre of the middle panel, and to load this disk with $27 \times 27 \times 12$-in. rectangular building-stones weighing about 914 lbs. each. These stones were placed one at a time by means of a derrick. The weight of the final load was about 11,882 lbs., concentrated on a disk $8\frac{1}{2}$ ins. in diameter or 0.394 sq. ft. in area. The resulting deflections as the stones were added one after another are shown by the following tabular statement:

Load, in Pounds.	Deflection.		Load, in Pounds.	Deflection.	
	Total.	Reduced.		Total.	Reduced.
914	$\frac{1}{32}$ in.	$\frac{1}{32}$ in.	7,312	$\frac{5}{8}$ in.	$\frac{35}{64}$ in.
1,828	$\frac{1}{16}$ "	$\frac{3}{64}$ "	8,226	$\frac{3}{4}$ "	$\frac{21}{32}$ "
2,742	$\frac{3}{32}$ "	$\frac{5}{64}$ "	9,140	$\frac{7}{8}$ "	$\frac{49}{64}$ "
3,656	$\frac{1}{8}$ "	$\frac{3}{32}$ "	10,054	$\frac{17}{16}$ "	$\frac{15}{16}$ "
4,570	$\frac{1}{4}$ "	$\frac{13}{64}$ "	10,968	$1\frac{3}{16}$ "	$1\frac{1}{16}$ "
5,484	$\frac{3}{8}$ "	$\frac{5}{16}$ "	11,882	$1\frac{7}{8}$ "	$1\frac{41}{64}$ "
6,398	$\frac{15}{32}$ "	$\frac{25}{64}$ "			

"At 5484 lbs. the concrete began to crack, and this cracking continued until the load reached 9140 lbs., when the lenses began to crack. The breakage continued until at 11,882 lbs. crushing and in general breaking of the concrete and glass took place. After the removal of the load the slab returned to a permanent deflection of $1\frac{1}{4}$ ins. In the table given above the reduced deflections given are the figures obtained by subtracting in each

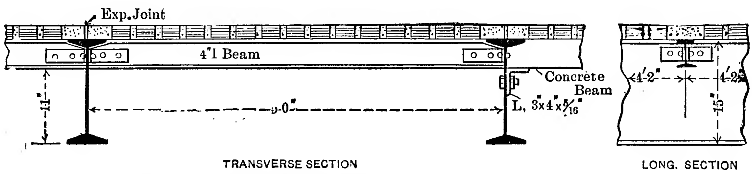


FIG. 83.—SECTIONS SHOWING VAULT-LIGHT FRAMING.
(Tucker & Vinton, Inc., New York City, Builders.)

case the deflection of the supporting beams from the total deflection given in the first column. While, as a result of the final load, the concrete and lenses were crushed as shown, the twisted rods were not broken, and the slab continued to support its load.

"Following the static-load test an impact test was conducted. In this test a stone weighing 914 lbs. was dropped 17 ft. and struck on one corner directly over one of the 5-in. beams. The effect of this blow was to deflect the beam permanently about 4 ins.; to pierce a 9×12-in. hub through the slab and bend but not break the steel rods, and to break about twenty lenses. The same stone was then dropped from a height of 19 ft. 4 ins., striking on its end, at the middle of one end panel. The effect of the blow was to smash a large hole through the concrete."

CONCRETE-STEEL SLABS, ARCHES, AND BARS.

THE following illustrations of slabs, arches, bars, etc., are taken from the catalogue of the St. Louis Expanded Metal Fireproofing Co.; A. L. Johnson, company engineer.

In Fig. 84 is shown a small mesh of expanded metal.

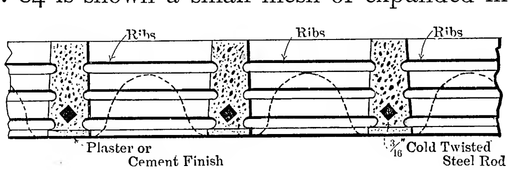


FIG. 84.—ENLARGED SECTION OF PORTION OF VAULT-LIGHT SLAB, SHOWING CONSTRUCTION.

In Fig. 85 is an illustration of a $1\frac{1}{4}$-in. square bar; net section, 1.07 sq. ins.; weight, 4.20 lbs. per foot. These corrugated bars are made from $1\frac{1}{4}$-in. to $\frac{1}{2}$-in. bar.

Floor-slabs.—System No. 1 is a long span system adapted to spans from 8 to 18 ft. in length. (See Fig. 85.) The thickness of the flat slab of concrete is taken from Johnson's Tables, the

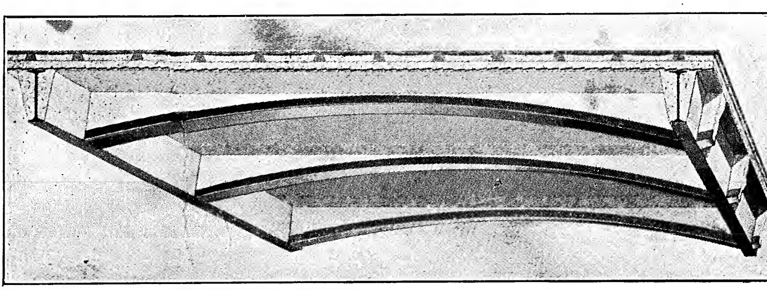


Fig. 85.—System No. 1. Golding Floor.

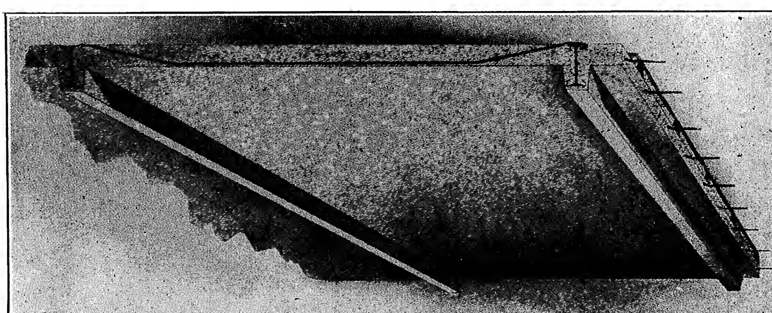


Fig. 86.—System No. 2. Corrugated Bar.

span being the distance between the ribs, usually about 4 ft. The size of the arch channels is obtained from the following formula:

$$A = \frac{Ldl^2}{8000r},$$

where A = area of channel in square inches;
L = total safe load in pounds per square foot;
d = distance between ribs in feet;
l = span in feet;
r = rise of arch channel in inches.

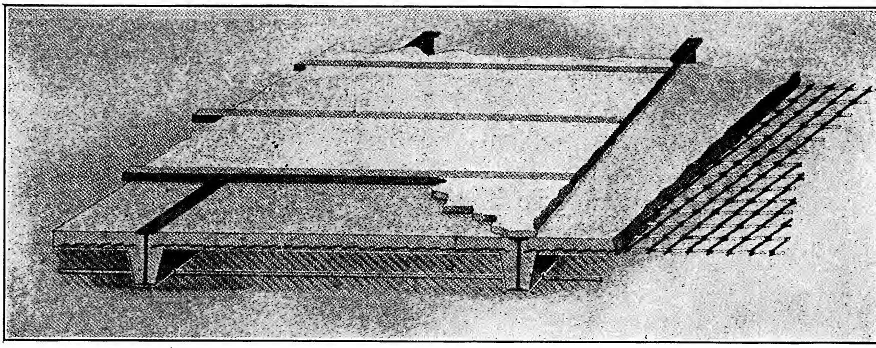


FIG. 87.—SYSTEM NO. 3. EXPANDED-METAL FLAT SLAB.

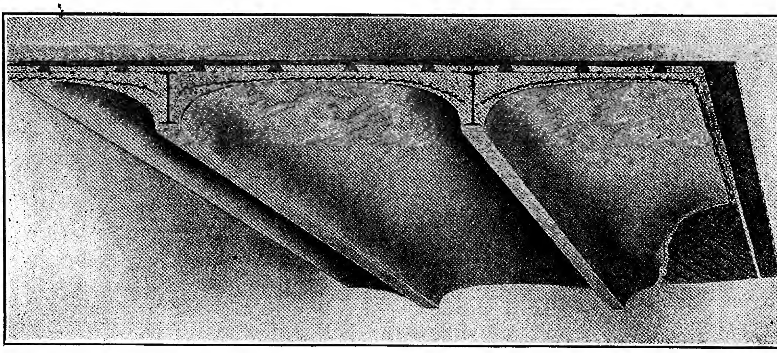


FIG. 88.—SYSTEM NO. 4. EXPANDED-METAL ARCH.
Suitable for spans up to 9 feet, where heavy loads are to be carried.

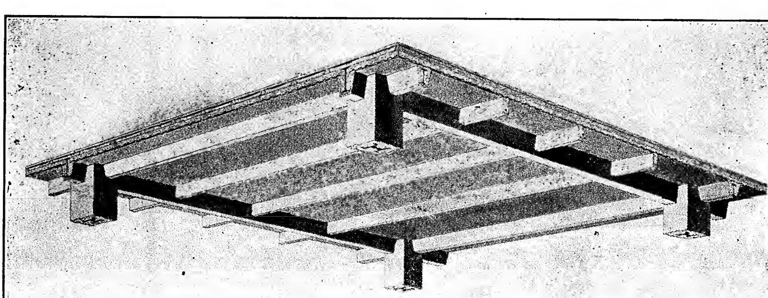


FIG. 89.—SYSTEM NO. 5. CORRUGATED BAR RIBS SUPPORTING EXPANDED-METAL ARCH.
Suitable for spans up to 30 feet in height.

System No. 2.—This is the best and latest construction for medium spans, from 8 to 14 ft. The longer spans, System No. 6, should be employed. Expanded metal has not sufficient strength for long spans, and corrugated bars should be employed. In Fig. 86 is shown the slab reinforced with the corrugated bar.

System No. 3, known as the expanded-metal flat slab, is shown in Fig. 87.

System No. 4.—In Fig. 88 the expanded-metal arch is shown.

System No. 5.—Fig. 89 is a representation of the long-span corrugated-bar construction.

BREAKWATER OF CONCRETE.

MANY of the lake harbors in the United States are formed by the construction of breakwaters. Most of these structures rest upon the clay and sand bed forming the bed of the lake. Any structure resting upon this material is certain to settle more or less, and also to have its alignment more or less changed. This consideration, together with the comparatively less cost of material, led to the simple timber-crib construction, the pockets of the crib being filled with broken stone, and covered with plank laid at the proper slopes. The entire structure, below and above the water surface, was mainly of wood. At Buffalo, N. Y., the longer cribs are 180 ft. in length, 36 ft. in width, and 22 ft. in height; the shorter cribs are 60 ft. in length. These are built and sunk in position end to end and independently. Any unequal settlement or displacement of these separate sections will not seriously impair the usefulness and effectiveness of the structure as a whole. The timber above the water surface will rot or otherwise be injured or destroyed by the force of waves in storms, and will require renewal from time to time, or above the water surface masonry or concrete walls could be built resting on the old crib below the water surface. After such a lapse of time there would be no danger of further settlement or displacement, and a masonry superstructure would not be subjected to the danger of cracking, splitting, or being totally displaced, as would have been the case had masonry been placed on the foundation of the crib in the beginning.

See the author's Civil Engineering, pages 500–525, for the general construction of timber cribs, and pages 693 and 694 for timber-crib dams.

At the present day, while timber cribs on pile foundations may be used, the superstructure is generally of masonry or concrete. Timber is not so plentiful and is more costly than it was a few years ago. A typical timber-crib breakwater to which is connected one with concrete superstructure is illustrated in Fig. 90.

Fig. 91 is a cross-section of a breakwater in Cleveland Harbor. The substructure consists of a timber crib, and the superstructure of concrete.

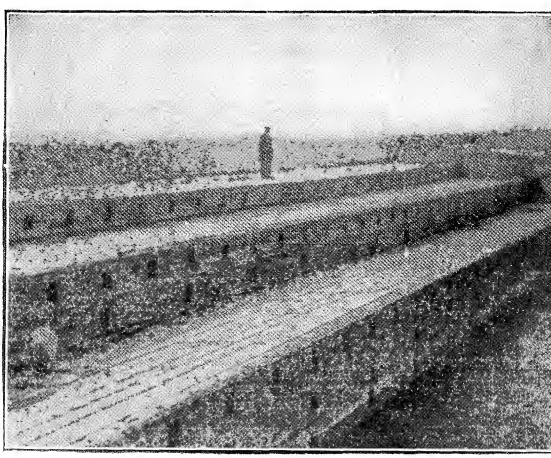


FIG. 90.—JUNCTION OF TIMBER-CRIB SUPERSTRUCTURE AND CONCRETE SUPERSTRUCTURE; SOUTH HARBOR BREAKWATER, BUFFALO.

In Fig. 92 the cross-section of the new breakwater at Buffalo, N. Y., is shown.

The old timber crib at Buffalo was badly wrecked during a storm. The velocity of the wind was reported to be 80 miles per hour. During this storm the water reached a height of 6.4 ft.

above mean lake level, subsequently falling to a line 3 ft. below, a total variation in water level of 9.4 ft. The waves dashing against the vertical wall of the structure rose to an estimated, height of 125 ft.; in falling on the structure they crushed the

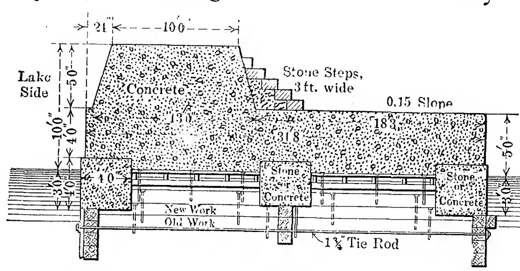


FIG. 91.—TYPICAL CROSS-SECTION OF CONCRETE SUPERSTRUCTURE FOR WEST BREAKWATER, CLEVELAND HARBOR.

largest timbers. The elevations and dimensions of the new breakwater appear in Fig. 92 and a general view of the breakwater showing the concrete blocks in place can be seen in Fig. 93. No further description seems necessary.

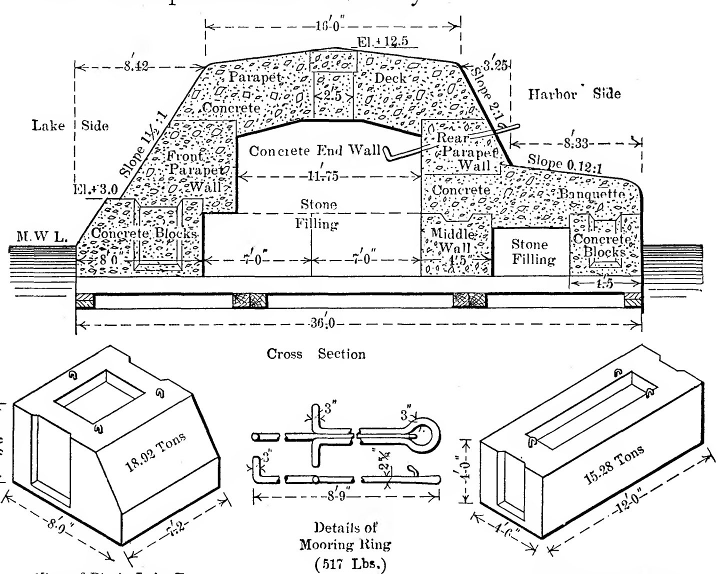


FIG. 92.—TYPICAL CROSS-SECTION OF CONCRETE-SHELL BREAKWATER FOR REPLACING TIMBER SUPERSTRUCTURE WRECKED BY STORM AT BUFFALO, AND ISOMETRIC VIEWS OF FACE BLOCKS.

An important feature is the construction of the concrete blocks, and also the proportion of the ingredients. Over the parapet walls and stone filling the deck is placed, 4.5 ft. thick at the centre and 3.5 ft. at the sides. Each section is provided with a manhole 2.5 ft. in diameter. This affords access to the interior, and allows the rubble stone to be introduced in case of settlement. It also serves as a vent for the escape of compressed air resulting from the rise and fall of waves, each manhole-cover having three large holes. The joggle-channels are filled with concrete as soon as blocks are placed. The

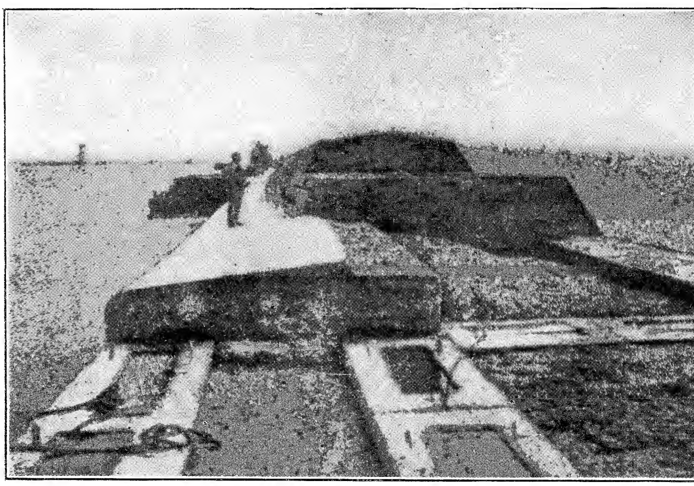


FIG. 93.—CONCRETE SUPERSTRUCTURE IN SUCCESSIVE STAGES OF CONSTRUCTION: CONCRETE BLOCKS; BANQUETTE; PARAPET WALLS, AND DECK.

concrete blocks are made in forms or moulds. Generally the ingredients of the concrete were mixed in proportion of 1 part cement, 1 part small gravel and sand, 2 parts sand grit, 4 parts unscreened broken limestone.

The proportions were varied so that all voids were filled with mortar and some excess mortar. The Engineer, Major T. W. Symons, U. S. A., says he "knows that the use of unscreened broken stone, run-of-the-crusher, is not in accord with the past engineering experience, but he believes that it is the best practice, and that engineers have for years been discarding from their concrete aggregates the best and most valuable parts of the broken stone. In 1882 the author of this book made extensive and elaborate trials with screened stone, and soon satisfied himself that the unscreened stone when used resulted in a much more uniform and homogeneous mass of concrete than possibly attainable with the screened stone; and in fact he goes farther and says it is difficult, if not impracticable, to make a homogeneous monolithic mass of concrete with screened stone. Anything from the size of a large grain of sand to the limit of 2 to 2½ inches diam, eter, mixed as it comes from the crusher, makes the best concrete. The impalpable dust is or should be removed. (See page 16.)

In proportioning the ingredients the unit of measurement was the bag of cement, containing 0.9 cu. ft. of cement and weighing from 95 to 100 pounds. The sand and gravel were measured in wheelbarrows having a capacity of 3.6 cu. ft. The broken stone was measured in a steel bucket to a volume of 14.4 cu. ft.

Each batch contained as follows:

4 bags of cement	3.6 cu. ft.
1 wheelbarrow load of small gravel	3.6 " "
2 wheelbarrow loads of sand grit	7.2 " "
1 bucket of screened broken stone	14.4 " "
Total	28.8 " "

The mixture was deposited in a receiving-chute, and when fully charged, the concrete was allowed to drop into the mixer. The mixer was a 4-ft. cube-mixer of the ordinary type, driven by a small upright engine.

The concrete was made quite wet, using from 2 to 2.5 cu. ft. of water to the batch.

The mixture was deposited in the moulds in layers of from 0.4 to 0.5 ft. in thickness. Fifteen to twenty batches were mixed in an hour—about 10 to 14 cubic yards. By working an ordinary spade up and down between the concrete and the forms, and tamping with a narrow rammer, a smooth-faced block was obtained. It was found that 28.8 cu. ft. of materials, measured separately, made 20 cu. ft. of concrete, or that it would require 38.88 cu. ft. of materials to make 1 cu. yd. of concrete.

5.4 bags or 1.35 barrels of cement	4.86 cu. ft.
1.35 wheelbarrows of small gravel	4.86 " "
2.70 " " sand grit	9.72 " "
1.35 buckets of broken stone	38 88 " "
1 cubic yard concrete	38.88 " "

The blocks were freed from the moulds in from 24 to 72 hours after placing, but, of course, were not handled for a longer period. For handling three bolts were moulded in the top face of the block.

The weight of the concrete was determined by displacement of water when scow was loaded with from 88 to 112 tons. Concrete 5 days old was found to weigh 152 lbs. per cubic foot. Other experiments showed about the same weight. It was also found that after 5 months there was a loss of weight of about 1 per cent.

The tops of the banquettes and parapets were finished off with a mixture consisting of 1 part cement, 2 parts sand, and 1 part gravel. This layer is about 0.3 ft. in thickness, and thoroughly tamped and kneaded into the coarse concrete; a smooth surface is obtained by means of straight-edges and wooden floats.

From records it was found that each batch containing 43.2 cu. ft. of the ingredients made about 28.5 cu. ft. of concrete in place, showing a shrinkage of 34 per cent.

*Cement Tests.**—Samples were taken from the bags, each sample containing from 4 to 5 lbs. cement. An average number was one sample for each 87 bags. Generally twelve briquettes were made from each sample, six neat cement and six standard sand briquettes, in the proportions of 1 cement and 3 sand. For the neat-cement samples there were used 30 oz. of cement and 6 fluid oz. of water; for the six sand briquettes, $7\frac{1}{2}$ oz. of cement, $22\frac{1}{2}$ oz. sand, and about $2\frac{1}{2}$ oz. water. In making the briquettes, the materials were mixed upon a glass slab and moulded in the usual manner. The moulded briquettes were subjected, by means of a weighted lever, to a pressure of 64 lbs., i.e., 16 lbs. per square inch of surface. The briquettes were kept in air until thoroughly

* See *Eng. News*, May 29, 1902.

set, then immersed in water, where they remained until tested. The Lehigh Portland cement, Allentown, Pa., was exclusively used.

Samples tested			613
Fineness, No. 100 sieve, per cent			94.1
Time of setting: initial, minutes			0.58
" " " final, "			276
Tensile strength in pounds per square inch.	Neat	7 days	687 lbs.
		28 "	775 "
		6 months	796 "
	Quartz sand 1:3	7 days	225 "
		28 "	324 "
		6 months	416 "

A number of briquettes of cement and stone-crusher sand made from limestone mixed in the proportions 1:3 gave the following results:

24 hours	7 days	14 days	21 days	28 days
72	375	396	458	567

pounds per square inch in tension.

Below are given the costs of construction of a number of breakwaters. The prices for materials are per unit placed in the work.

Dunkirk Breakwater.—Shell concrete superstructure; cost per linear foot of superstructure, \$41.12; concrete blocks, per cubic yard, \$6.73; concrete in place, per cubic yard, \$6.73; concrete manhole-covers, \$14 each; filling-stone, per cubic yard, \$1.19.

Old Breakwater, Buffalo.—Solid concrete superstructure; section built in 1889, per linear foot of superstructure, \$110.19; concrete in place, per cubic yard, \$9.19; section built in 1891 per linear foot of superstructure, \$108.32; concrete in place, per cubic yard, \$8.21.

Old Breakwater, Buffalo.—Shell concrete superstructure: Per linear foot of superstructure, \$65.00; concrete blocks, per cubic yard, \$6.64; concrete in place, per cubic yard, \$6.64; filling-stone, per ton, \$0.66; concrete manhole-covers, \$1.25 each; mooring-rings, \$30 each.

North Breakwater, Buffalo.—Shell concrete superstructure: Per linear foot of superstructure, 36-foot section, \$56.34; concrete blocks, per cubic yard, \$7.10; concrete in place, per cubic yard,

$5.65; filling-stone, per ton, $0.69; concrete manhole-covers, $3 each; mooring-rings, $50 each.

South Harbor Breakwater.—Shell concrete superstructure: Per linear foot of superstructure, $99.44; concrete blocks, per cubic yard, $10; concrete in place, per cubic yard, $9.40; filling-stone, per ton, $0.80; concrete manhole-covers, $5 each; mooring-rings, $55 each.

The costs per linear foot of superstructure are exclusive of engineering and office expenses, and also exclusive of removing old timber superstructures and repairing crib foundation in the case of the Old Breakwater concrete superstructure and the South Harbor Breakwater.

The cost of this removal and preparation of foundations on the several structures was as follows:

Old Breakwater.—Solid concrete section, per linear foot of breakwater, $5.42. Shell construction section, per linear foot of breakwater, $7.26.

South Harbor Breakwater.—Per linear foot of breakwater, $8.

THE NEW BREAKWATER, WELLAND CANAL ENTRANCE, ONTARIO, CANADA.

The Welland Canal connects Lake Erie with Lake Ontario; its length is 26¾ miles. Lake Erie is 573 ft. above sea-level and Lake Ontario is 246 ft.—a difference of level of 327 ft., which is overcome by 25 lift-locks. The entrance to the canal is protected by two piers. The canal basin is an area 2300 ft. in length and from 200 to 300 ft. in width. The piers extend in a generally southerly direction, and are of timber-crib construction, 2108 ft. in length and 30 ft. in width.

On the prolongation of the west pier, and at a distance of 2300 ft. from the outer extremity of the pier, is located the east end of the new breakwater. The breakwater extends in a general direction a little north of east for a distance of 5700 ft. to the shore-line of Lake Erie. The design and construction of the breakwater are clearly shown in Figs. 95, 96, 97, and require little description. There are 48 separate cribs, each 100 ft. long and 25 ft. wide, and of varying heights according to the depth of water.

At the shore end the breakwater connects with an embankment 800 ft. long, and at the other end is an outer block 100 ft. long and 60 ft. wide.

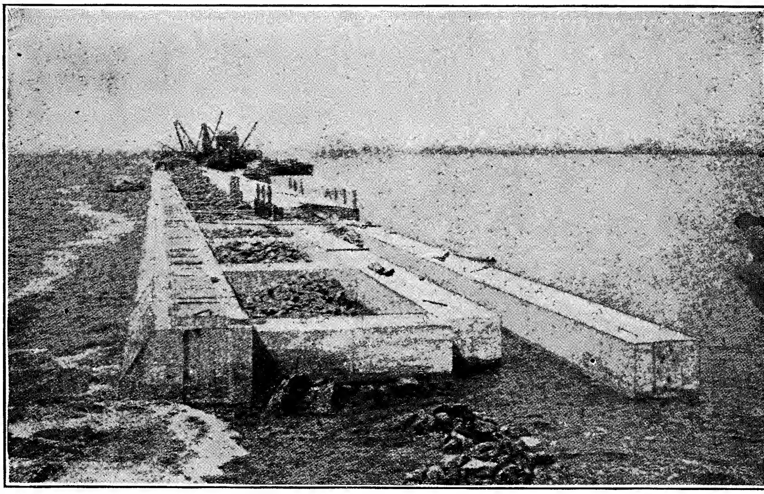


FIG. 94.—VIEW OF CONCRETE BLOCKS IN PLACE FOR BREAKWATER SUPERSTRUCTURE.

The cribs are built of spruce, hemlock, white or red pine, and tamarack up to 3 ft. below low water. Above that level

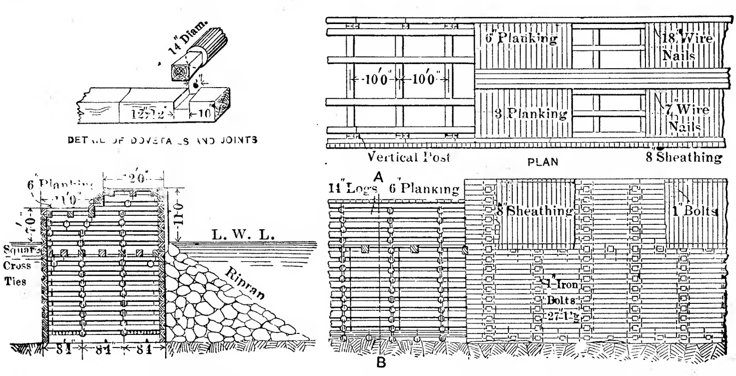


FIG. 95.—CROSS-SECTION, ELEVATION, AND PLAN OF PORTION OF BREAKWATER.

white o red pine and tamarack are used, except that the sheathing is of rock elm.

The lower seven courses of timber in cribs were laid on ways and launched; the remainder of the work was done floating. The cribs are floated to their proper positions and sunk by means

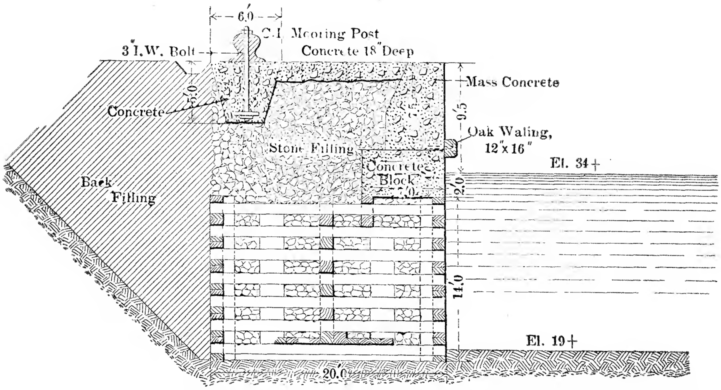


FIG. 96.—SECTION OF CONCRETE RETAINING-WALLS FOR THE WELLAND CANAL BASIN.

of weights placed on top until the cribs rest on the natural lake bottom. The shore end is built of stone and is 800 ft. in length,

FIG. 97.—END VIEW OF CONCRETE RETAINING-WALL, SHOWING CONCRETE BLOCKS RESTING ON SUBMERGED CRIBS AND MASS CONCRETE SUPERSTRUCTURE.

top width 25 ft., side slopes 1 on $1\frac{1}{2}$. The faces are built of larger stone, and the filling-stone dumped in between. The top is finished off with broken stone at a height of 7 ft. above low water.

The basin proper is lined with retaining-walls consisting of concrete and stone filling resting on timber cribs, which, after being sunk, are also filled with broken stone (see Fig. 96). The concrete blocks are 7 ft. long, $4\frac{1}{2}$ ft. wide, and 4 ft. high. The blocks are moulded with joggles and holes for the ends of the hooks. After being placed the joggles are filled with concrete. Over the blocks is placed mass concrete of the proper height

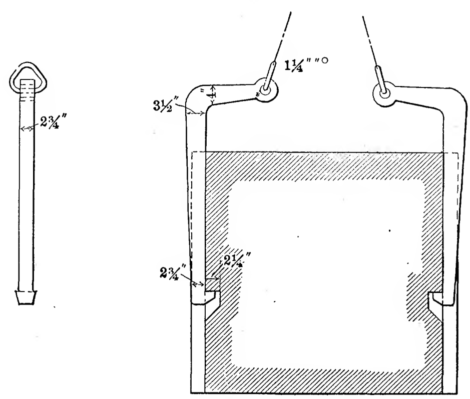


FIG. 98.—HOOKS FOR LIFTING CONCRETE BLOCKS.

and cross-section. The concrete is made of 1 part Altas Portland cement, 2 parts sand, and 4 parts broken stone.

The blocks are lifted by means of hooks as shown in Fig. 98.

SEWER FOUNDATIONS.

THE foundations for sewers depend largely upon the character of the bottom of the trench. They may be laid on the natural soil if compact, dry, and hard. They may be bedded in a layer of sand; in other cases on a bed of concrete or a flooring of plank In case of very soft silt or quicksand it may be necessary to drive piles to a greater or less depth, which are capped after being cut off at a level at or near the bottom of the trench, upon which a flooring of plank is placed, or concrete may be rammed over and around the top of the piles, upon which the sewer is laid. Strong and close sheathing may be required in excavating in soft materials.

It is especially important that provisions should be made to prevent any unequal settlement along the pipe-line, otherwise material will be deposited affecting materially the flow of sewage, if not ultimately resulting in completely filling and choking the sewer.

If it is required to build a sewer across a river, it will generally be advisable to build a double-wall coffer-dam in sections and, after making the necessary excavation in the bed of the river, to put down a bed of concrete properly rammed, and upon this the pipe is laid or the sewer constructed; concrete is then placed and rammed over and around the sewer. Many sewers are now constructed entirely of concrete.

THE SIXTY-FOURTH STREET OUTLET-SEWER, BROOKLYN, N. Y.

THE diameters of these sewers vary from 66 ins. to 15 ft. Full specifications and drawings are found in *Eng. News*, Jan. 1, 1903. About 964 ft. of the triple sewer structure is brick masonry enclosed in heavy walls of concrete on each side and a concrete roof-covering, the whole being carried on a timber platform supported on piles. The specifications are full and complete. The bearing piles are to be not less than 45 ft. in length, and are to be driven vertically full length from the bulk-head line to 25 ft. inland.

The piles shall not be less than 12 ins. in diameter at large end, shall be properly sharpened, and shod with iron shoes when required. They are to be of good, sound pine or spruce, free from shakes. They are to be driven with a hammer weighing not less than 2000 lbs., and are to be driven to a solid bottom, or until the penetration shall not exceed 1 in. under the blow of the hammer falling 15 ft., or until the pile has reached a depth of 80 ft.

Should the resistance to penetration be greater than given above at the depths prescribed, the bearing-piles for this distance shall be driven with the water-jet until 40-ft. lengths are obtained under cut-off. Other bearing-piles spaced generally 4 ft. centres, longitudinally, as shown on plans, are driven vertically. Spur or brace piles are also to be driven at an angle of 20° to the vertical. All spur piles are to be 46 ft. in length.

Cement and Mortar.—All mortar is to be composed of 1 part of fresh-ground hydraulic cement of the best quality and 2 parts of clean, sharp sand entirely free from loam. It is to be carefully and thoroughly mixed while dry, and a sufficient quantity of water is to be afterward added to make it a good consistency. The mortar is to be mixed in no greater quantity than is required for the work in hand. No mortar is to be used after taking a set, and is not to be used if retempering is necessary. All cement must be delivered on the ground in well-coopered barrels, and not in bags nor in bulk, and the contractor will be required to furnish the engineer with full facilities for testing the same; all cement found to be of improper or inferior quality must be immediately removed from the work.

All cement, except Portland cement, shall be 90 p. c. fine when tried with a sieve of 2500 meshes to the square inch, and shall be subject to either of the following tests: Neat cement, made into testing-blocks, after 30 minutes' exposure to air and 24 hours' immerson in water shall withstand a tensile stress of 75 lbs. per square inch for Rosendale and 160 lbs. for Portland cement. Portland cement, after exposure in air for one day and in water for six days, shall withstand a stress of 350 lbs. per square inch, and when mixed with 2 parts of sand, after one day in air and six days in water, a stress of 125 lbs. per square inch.

Concrete.—All broken stone shall be "machine-broken," of hard, sound granite, trap-rock, or silica sea-washed gravel, to be approved by the engineer, and of a size that will pass through a 1¼-in. ring; it must be free from dust and dirt, and, if ordered, must be thoroughly screened and washed before being used; the cement and sand must be up to the standard prescribed for mortar. Concrete shall be made in the proportions of 1 part, by measure, of cement, two of sand, and five of broken stone, except the concrete specified for the outlet-sewer. The cement and sand shall first be made into a rather stiff mortar, then the stone will be added, and the whole mass is quickly and thoroughly mixed in a box made for that purpose, by turning it over three times with shovels. Concrete must be lowered into the trench in boxes or buckets, and will in no case be allowed to be cast in by shovels from the surface; it shall be deposited in layers not less than 4 nor more than 9 ins. in thickness, and must be settled in place by ramming sufficiently to flush the mortar to the surface. When connection is to be made with any layer, set or partially set, the edge of such layer must be broken down and all loose material removed, the surface swept off and thoroughly wetted, so as to make the joint fresh and close.

Concrete for Outlet-sewer.—The sewer near the bulkhead line shall be built entirely of concrete. The inner faces of the triple sewers to be faced with an 8-in. layer of concrete, mixed in the proportions of 1 part of Portland cement, $\frac{1}{2}$ part of sand, and $1\frac{1}{2}$ parts of granite-dust. All other concrete in this length to be 1 Portland cement, 2 sand, and 4 broken stone. (*Eng. News.*, Jan. 1, 1903.)

CONCRETE-FILLED COLUMNS.

It is generally admitted that painting iron is an uncertain and ineffective means of preservation. The painting of interior surfaces is impracticable, or at least very difficult.

It is claimed that rusting or corrosion can only appear in the presence of moisture, air, and carbon dioxide. Iron will not rust in dry air, or in water containing a free alkali, an alkaline earth, or an alkaline carbonat . Portland cement being strongly alkaline is therefore considered a good protection against corrosion. The easiest method of applying this preservative is by filling the column with concrete, and also the space between the column and the fire-proof casing. It is recommended to mix the concrete with whitewash instead of clean water. The cóntact between the steel and cement or concrete should be full and perfect. Protecting the column outside and inside with mortar or concrete renders painting unnecessary. If the above statements are true, it is probable that existing steel columns will be to a greater or less extent filled with concrete. And in future structures the tendency will be to build concrete columns in which will be embedded iron columns or iron rails.

Other advantages are the greatly increased stiffness and bearing capacity of columns filled with concrete, and further the greater safety in case of fire.

At the new Government Printing Office the columns are composed of channels placed back to back, latticed on with cover-plates according to the load. These were fireproofed with 4 ins. of brickwork, the whole enclosed open space being filled with concrete. The proportions used were 1 cement, 4 sand, and 9 broken stone not over 1 in. diameter, and whitewash used in mixing. The concrete is mixed rather wet and shovelled into the open space, the compacting resulting from the fall. If the fall is too great, or concrete too wet, the fire-proof covering may burst under the hydrostatic pressure.

Columns formed of 6-in. extra-heavy pipe and 21 ft. long, designed to carry a load of 25 tons, were subsequently filled with a concrete of the proportions 1, 3, 6, and loaded with 63 tons, which would not have been a safe working load on the unfilled columns.

CONCRETE FOUNDATIONS.—MONIER SYSTEM.

THE Monier system has recently been applied to the construction of large bins or tanks. The foundation is constructed of concrete 3 ft. in thickness, forming a continuous floor 66×66 ft. A netting of $\frac{5}{8}$-in. round steel rods is tied at intersections with No. 18 wire, forming a metal layer of 9-in. mesh. This is embedded in the concrete about 5 ins. above the base.

Upon this concrete base a series of concrete piers, 12 ft. 6 ins. high and 1 ft. 10 ins. thick, and varying in length from 3 ft. 5 ins. to 7 ft. 10 ins. These rest upon $\frac{1}{2}$-in. steel plates embedded in the concrete floor about 15 ins. below the upper surface. In the smaller piers four steel rails are placed and in the larger six steel rails, all set vertically and connected by spacing-bars riveted to them. The piers are capped with steel plates. Upon the piers are placed concrete beams 4 ft. wide with steel girders 15 ins. deep, with vertical openings at intervals for the discharge spouts.

Horizontal lines of steel rods run through each girder near the top, and four other lines bent to form truss-rods, with sheets of wire netting on each side of each pair of rods. Upon this foundation four cylindrical tanks 53 ft. 6 ins. high and 25 ft. diameter spaced 29 ft. center to centre, are placed. The bases of the tanks, resting on the system of girders, are 13 ft. 9 ins. above the level of the concrete floor.

The walls of the tank are of concrete 7 ins. in thickness near the bottom and diminishing to 5 ins. near the top. A netting of No. 9 wires electrically welded with rectangular meshes 1×4 ins. is embedded in the walls. Around the netting, alternating inside and outside, are placed rings of iron rods tied to the netting by wire. These rods vary from 1 in. diameter at bottom of tanks to $\frac{3}{8}$ in. at top.

The top is finished with a 5-in. Z-bar ring. The cylinders have a conical roof 2 ins. thick, with opening for a manhole and spout. The bottom is made conical. There are eight discharge-openings, 15×48 ins., in the annular space between the base of the cone and the walls of the tank. The conical bottom is 4 ins. thick, reinforced with rods and netting. Its diameter at base is 22 ft. The concrete in the foundation is composed of 1 part Portland cement, 3 parts coarse sand, and 4 parts broken stone.

The walls of the tanks are made of concrete consisting of 1 part Portland cement and $3\frac{1}{2}$ parts sand, no stone being used. The concrete was mixed moderately wet and rammed in wooden forms. The forms were constructed of 3-in. plank 3 ft. long, kept together by horizontal angle-irons supported by vertical angle-irons clamped together above the wall. The concrete was placed and rammed between the forms, which were raised in 45° sections, 28 ins. high, every 24 hours.

For further details and description of operating machinery, see *Engineering News*, Dec. 11, 1902, from which the above description is taken.

(See article by Capt. Sewell in *Eng. News*, Oct. 23, 1902.)

THEORY OF THE STRENGTH OF BEAMS OF REINFORCED CONCRETE.

IN *Engineering News* of February 27, 1902, is found a full discussion of the strength of reinforced concrete beams by W. Kendrick Hatt. Mr. Hatt adopts as the basis of his computations the results of the experiments of M. Considère and confirmed by some experiments of his own. He says: "It may be said that there is no widely accepted method of computation for the strength of concrete reinforced with steel; furthermore, the elements of the strength of the constituent parts are not well ascertained. The truth of the analysis must then be tested by further experiment."

In the following analysis the units are assumed to be correct. The coefficient of elasticity of steel $E_s = 30{,}000{,}000$ lbs. per square inch; elastic limit 40,000 to 50,000 lbs. per square inch.

From the Watertown Arsenal tests on 12-in. cubes of Portland cement and broken-stone concrete mixed dry, the following units are obtained:

SAYLOR'S CEMENT CONCRETE OF 1:2:4.

Coefficient of elasticity, E_c	1,667,000	3,461,000	4,500,000
Load	1,000	1,000	1,000
Crushing strength, C_c	1,944	2,200	3,500

CONCRETE 1:3:6.

E_c	1,956,000	3,750,000	2,812,000
Load	1,000	1,000	1,000
C_c	2,308	2,500	3,500

The above values are in pounds per square inch.

The coefficient of elasticity of gravel concrete is greater than that of stone concrete, while that of cinder concrete is less. Wet mixtures have a modulus much lower than that of plastic mixtures. "The modulus of elasticity is a very important element in the calculation of the strength of reinforced-concrete beams, and should be obtained from the actual material used in each case."

It is further assumed that the reinforced-concrete beam supports an elongation ten times that of the plain concrete and mortar without sign of fissure. Concrete and mortar plain breaks with an elongation of less than 1 in 10,000, while that of reinforced concrete will support an elongation of 1 in 1000. This 1 in 1000 is nearly the elongation of steel at its elastic limit. The stress in the steel reinforcing material may be near its elastic limit, and yet the concrete in tension may still be carrying load without cracking. The tensile strength of concrete is about $\frac{1}{10}$ its compressive strength; therefore for 1:2:4 concrete at the age of three months it will be (from the table above) 220 lbs. per square inch.

A material like concrete or mortar will not be perfectly elastic even for very light loads. Hooke's law that the stress increases directly as the strain does not hold; the analysis must be based on the actual stress-strain curve, and we must compute the ultimate or breaking load of the beam, and not, as for steel beams, the elastic strength.

Assumptions.—In deriving a formula for the moment of resistance of the beam, it is assumed:

1. The cross-sections of the beams remain plain surfaces during flexure. This means (see Fig. 99) that two sections

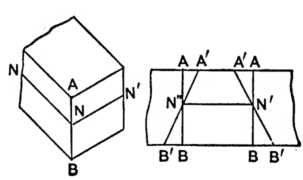


FIG. 99.—DIAGRAM ILLUSTRATING POSITION OF NEUTRAL AXIS.

originally parallel will rotate to $A'B'$ after impositions of the load. The neutral surface will be somewhere between the upper compressed fibres and the lower stretched fibres. Its intersection NN' with the cross-section of the beam is called the neutral axis. The locations of this neutral axis is a part of the computation.

2. The applied forces are perpendicular to the neutral surface.

3. It is assumed that the presence of material surrounding any elementary fibre will not modify the effect gate or be compressed just as if it were under that load by itself in a testing-machine.

4. There is no slipping between the faces of the wire and the surrounding concrete.

5. The elastic limit of the concrete is exceeded in both the compression and the tension flange.

6. There are no initial stresses due to shrinkage or expansion of the concrete while setting.

In Fig. 100 the stress-strain diagram is shown; the curve of variation of the compressive stress is assumed to be a parabola, and that of the tensile is curved for a short distance below the neutral axis and then remains uniform to the lower fibre. In Fig. 101 is shown Considère's modification of Fig. 100. While the diagram adopted in the following analysis is shown in Fig. 102,

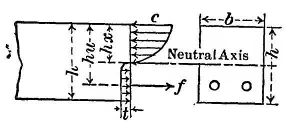


FIG. 100.

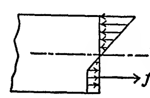


FIG. 101.

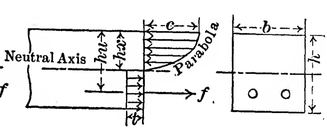


FIG. 102.

FIG. 100.—DIAGRAM ILLUSTRATING STRESSES ON SECTION OF REINFORCED-CONCRETE BEAM.
FIG. 101.—CONSIDERE'S MODIFICATION OF FIG. 100.
FIG. 102.—HATT'S MODIFICATION OF FIG. 100, USED IN ANALYSIS.

in which the variation of compressive stress is represented by the ordinates of a parabola with its vertex at the point of highest strength, the tensile strength of the concrete is assumed uniform from the neutral axis to the extreme lower fibre, and the tensile resistance of the metal reinforcement is represented by the arrow f.

ANALYSIS.

Referring to Fig. 102, h and b are respectively the depth and width of a rectangular beam. The following explains the significance and meaning of the terms employed:

hx = distance to the neutral axis from extreme upper fibre;
hu = " " reinforcement from upper compressed fibre;
F = area of concrete;
F' = " " steel;

$$p=\frac{F'}{F}\left(\text{usually from } \frac{1}{75} \text{ to } \frac{1}{100}\right);$$

x, u, and p are ratios;

E_c = modulus of elasticity of concrete;
E_s = " " " " steel;
f = unit stress on steel reinforcement;
c = " compressive stress in outer fibre of concrete;
t = " tensile stress in outer fibre of concrete.

Referring to Fig. 102 we may derive the following relations:

Area in compressive diagram $=\frac{2}{3}chx$;
Total force on compression flange $=\frac{2}{3}chxb$;
Moment of compression about neutral axis $=\frac{2}{3}chxb\times\frac{5}{8}hx$;
$\frac{5}{8}hx$ = distance of centre of gravity of the parabolic segment from the neutral axis;
Total tensile force on concrete $=th(1-x)b$;
Moment of tension about neutral axis $=th(1-x)b\frac{h-hx}{2}$;
Tension on steel $=F'f=phbf$;
Moment of this tension about neutral axis $=phbfh(u-x)$.

Since the total tension acting to the right must equal the total compression acting to the left, we have:

$$\tfrac{2}{3}chxb=th(1-x)b+phbf,$$

or

$$\tfrac{2}{3}cx=t(1-k)+pf. \quad \ldots\ldots \quad (1)$$

The moment of resistance of the section about the neutral axis is

$$M=\tfrac{2}{3}chxb\times\tfrac{5}{8}hx+th(1-x)bh\frac{1-x}{2}+pbhfh(u-x)$$

$$=bh^2\left(\frac{5}{12}cx^2+t\frac{(1-x)^2}{2}+pf(u-x)\right). \quad \ldots\ldots \quad (2)$$

Modulus of elasticity $E_c=\frac{\text{unit stress}}{\text{unit strain}}=\frac{c}{e_c}$, where e_c = unit compression on upper face of beam. Similarly, $E_s=\frac{f}{e_s}$.

Since the sections of the beam are assumed to remain plane surfaces,

$$\frac{e_c}{e_s}=\frac{hx}{h(u-x)}, \text{ or } \frac{c}{f}=\frac{E_c}{E_s}\cdot\frac{x}{u-x}. \quad \ldots\ldots \quad (3)$$

Substituting f from (3) in (1),

$$t(1-x)+pc\frac{E_s}{E_c}\frac{u-x}{x}=\tfrac{2}{3}cx. \quad . \quad . \quad . \quad . \quad . \quad (4)$$

These are the equations needed to compute the strength of a reinforced-concrete beam. From eq. (4) we may obtain the distance to the neutral axis (after having assumed the tensile and compressive strength and modulus of elasticity of the concrete, the location and number of the reinforced shapes, with the modulus of elasticity of the steel). Having found x, the value of f is computed from eq. (3), and finally the resisting moment of the section is found from eq. (2).

Neglecting the tensile strength of concrete, and assuming the metal reinforcement as affording the whole tensile resistance, eqs. (1), (2), and (4) become (see Fig. 104)

$$\tfrac{2}{3}cx=pf, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

$$M=bh^2[\tfrac{5}{12}cx^2+pf(u-x)], \quad . \quad . \quad . \quad . \quad . \quad (6)$$

$$p\frac{E_s}{E_c}(u-x)=\tfrac{2}{3}x^2. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

Neglecting the tensile strength of concrete and considering the compression diagram to be a straight line, the corresponding equations become

$$\tfrac{1}{2}cx=pf, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

$$M=bh^2[\tfrac{1}{3}cx^2+pf(u-x)], \quad . \quad . \quad . \quad . \quad . \quad (9)$$

$$\tfrac{1}{2}x=p\frac{E_s}{E_c}\frac{u-x}{x}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

Example.—To find the strength of a reinforced-concrete beam 8×12 ins. in section and 10 ft. in length.

$p=\frac{1}{80}=$ two $\frac{7}{8}$ in. rods.

$E_c = 2{,}000{,}000$; $E_s = 30{,}000{,}000$; tensile resistance $t = 200$; compressive resistance $c = 2800$.

Referring to Fig. 103, and using two reinforcing-rods 2 ins. from lower face of beam, $x = 0.388$; $hx = 4.97$ ins.; $f = 48{,}200$ lbs.

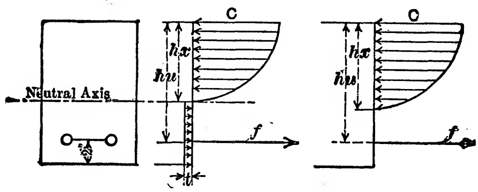


FIG. 103. FIG. 104.

per square inch; $M = 450{,}000$ in.-lbs.; centre load $= 15{,}000$ lbs.

Referring to Fig. 104, in which no allowance is made for tensile strength of the concrete, the reinforcing metal is assumed to bear all the tension, and instead of using 2800 lbs., the ultimate compressive resistance, allowing a factor of safety of 4, $c = 700$ lbs., we have

$$\tfrac{2}{3}cx = pf,$$
$$p\frac{E_s}{E_c}(u - x) = \tfrac{2}{3}x^2.$$

Eliminating p,

$$x = \frac{\frac{c}{f}\frac{E_s}{E_c}u}{\frac{c}{f}\frac{E_s}{E_c} + 1}.$$

This value of x multiplied by the total depth gives the distance to the neutral axis from the compressed side.

This value substituted in (1) gives

$$p = \frac{2}{3}\frac{c}{f}\frac{\frac{c}{f}\frac{E_s}{E_c}u}{\frac{c}{f}\frac{E_s}{E_c} + 1}.$$

This value of p multiplied by the cross-sectional area of concretes gives area of steel reinforcement required.

Substituting stress in steel, $f=16{,}000$ lbs. per square inch; $c=700$; $E_c=2{,}000{,}000$; $E_s=30{,}000{,}000$; and $u=\frac{5}{6}$ depth of beam = distance from extreme fibre in compression to the centre of gravity of the reinforcement.

Taking moments about the centre of gravity of the compressive parabola, which is $\frac{3}{8}hx$ from extreme fibre in compression, the moment of resistance of the section is

$$M=F'h[f(u-\tfrac{3}{8}x)].$$

With the above values of f, u, and x, the quantity in brackets becomes a constant, and

$$M=11{,}352F'h.$$

The amount of steel required varies directly as the depth of the section, and the moment of resistance varies directly as the square of the depth. Having solved one section, any other, under the same conditions or assumptions, may be readily solved.

PROPERTIES OF CONCRETE-STEEL BEAMS.

Depth of Section in Inches.	Centre of Gravity of Steel from Tensile Side in Inches.	Area of Steel in Sq. Ins. per Foot of Width.	Moment of Resistance in Ft.-lbs. per Foot of Width.
6	1	0.69	3,935
8	1.33	0.92	6,996
10	1.66	1.15	10,932
12	2.00	1.38	15,743
14	2.33	1.62	21,426
16	2.66	1.85	27,986
18	3.00	2.08	35,421
20	3.33	2.31	43,728
22	3.66	2.54	52,961
24	4.00	2.77	62,971

In the above table the width or breadth of the beam is assumed to be 1 ft., unit tensile stress in steel 16,000 lbs. per square inch, and the unit compressivve stress in concrete 700 lbs per square inch; these being safe working loads. (See article in *Eng. News*, May 15, 1902, by Louis F. Brayton.)

While the resulting formulæ in the foregoing solutions of problems in connection with reinforced concrete are not difficult to apply, the analyses leading to them are cumbersome and not readily understood by practical men. The assumed data and assumptions are admittedly unreliable, and consequently the results at best are only approximate. The author has sought some simple and tangible method giving results at least as satisfactory from a practical point of view. He has found such a method in *Eng. News*, Feb. 11, 1904, by Frank L. Batchelder. It is a mere adaptation of the usual formulæ for flexure in any rectangular-shaped beams.

Let b = width in inches; d = depth in inches; d_1 = depth of reinforcement below top of beam.

Ultimate strength of concrete in tension = 200 lbs. per sq. in.
" " " " " comp. = 2000 " " " "

Minimum value of elastic limit in steel in tension = 50,000 lbs. per sq. in.

The moment of resistance of a plain concrete beam of rectangular section is

$$\tfrac{1}{6}\,200bd^2,$$

and if by any means the full value of the compressive resistance of the beam can be developed, the resisting moment is

$$\tfrac{1}{6}\,2000bd^2.$$

The steel must therefore carry a resisting moment equal to the difference, or

$$\tfrac{1}{6}\,1800bd^2.$$

This divided by d_1 gives the ultimate stress the steel reinforcement must carry. This stress divided by 50,000 gives area of metal reinforcement. These are ultimate units. For safe working values use the usual factors of safety; that is, 50 instead of 200 for concrete in tension, 500 instead of 2000 for concrete in compression, and 12,000 to 16,000 instead of 50,000 for steel. The above assumes the neutral axis to be at the centre of the depth of the beam.

MATERIALS REQUIRED IN MAKING DIFFERENT CLASSES OF CONCRETE FOR CONNECTICUT AVENUE BRIDGE, WASHINGTON, D. C.

CLASS A. 1:2:4.28 CONCRETE.

4 bags = 1 bbl. vulcanite cement = 378.25 lbs.

= 4.5 cu. ft. 9 cu. ft. of sand 20.25 " " " stone	Yielded 21.4 cu. ft. concrete when rammed in place.

With 1 bbl. cement:

1:2½:6 concrete. 11.25 cu. ft. sand and 27 cu. ft. stone yielded 27.66 cu. ft. concrete when rammed in place.

1:2½:3:3 (3 gravel and 3 stone). 11.25 cu. ft. sand, 13.5 gravel, and 13.5 stone yielded 27.66 cu. ft. of concrete when rammed in place.

1:3:10 gravel concrete. 13.5 cu. ft. sand and 45 cu. ft. gravel yielded 45 cu. ft. concrete when rammed in place.

REINFORCED-CONCRETE RETAINING-WALLS AND DAMS.

THE uses to which reinforced concrete is now being put, the extravagant claims made, and the number of crude if not false theories advanced, have been thrust upon engineers so rapidly that it is difficult to form any well-digested opinions or conclusions. Old theories and formulæ must be abandoned or greatly modified. The retaining wall is not looked upon as a body or block that may give way by overturning, but is treated as a cantilever beam acted upon by a uniformly varying pressure, otherwise it is difficult to find any advantage due to the reinforcement.

In *Engineering News*, March 9, 1905, is found the latest development in the construction of high reinforced-concrete walls. The design of the walls is based upon the Johnson system of reinforcement. The concrete used is of 1:3:6 composition.

In the formulæ used the ultimate unit values are: coefficient of elasticity of concrete in compression 3,000,000 lbs. per square inch, of steel 29,000,000; elastic limit of steel corrugated bars 50,000 lbs. per square inch; ultimate strength of concrete in compression 2000 lbs. per square inch, and in tension 200 lbs. per square inch.

Mr. C. F. Graff, the writer of the article, says:

The face and base of the wall have been considered as com-

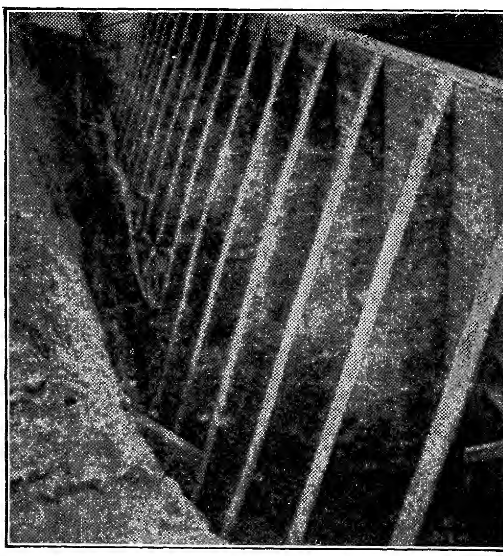


FIG. 105.—VIEW SHOWING REAR OF WALL.

posed of independent beams lying side by side, and thus introducing an unknown factor of safety, inasmuch as there is really a floor-slab action. The weight of loose earth was assumed to be 100 lbs. per cubic foot; the weight of the wall itself was neglected in the design. The lateral earth pressure behind the wall was found by Coulomb's theory, and multiplied by $\frac{3}{2}$, giving a factor of safety of about 3 against overturning, as values found by this theory are approximately twice too large by experiment. Where this wall is high, there is a solid bank as backing, and it is improbable that the thrust will approach that computed. The arrangement, size, and number of corrugated bars were carefully calculated. The front, rear, sloping face, and base were formed of bars encased in concrete, thus forming a series of ribs

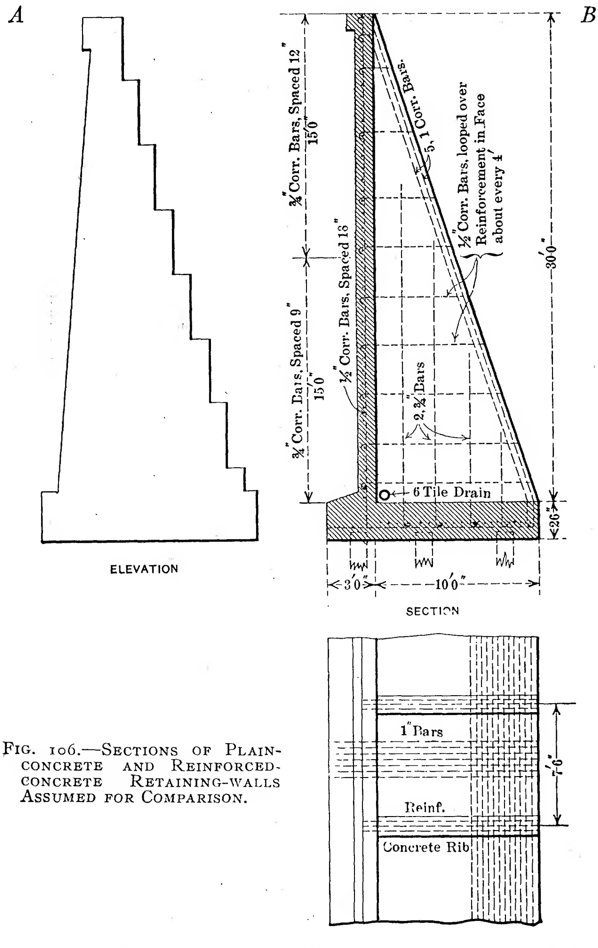


Fig. 106.—Sections of Plain-concrete and Reinforced-concrete Retaining-walls Assumed for Comparison.

spaced 7.5 ft. apart. On the front face these ribs were connected by a series of reinforcing-bars which were also encased in concrete. A view of the completed wall is given in Fig. 105, and in Fig. 106, *B*, is shown a vertical section along or through one of the ribs; in Fig. 106, A, is shown the standard plain-concrete wall having the "rule-of-thumb" thickness at base of $\frac{4}{10}$ the height.

Take a skeleton section of the rib, showing the front face and base-slab (see Fig. 107), and consider one wall foot. Assume w the weight of 1 cu. ft. of earth $=100$ lbs.; height, h, of the wall

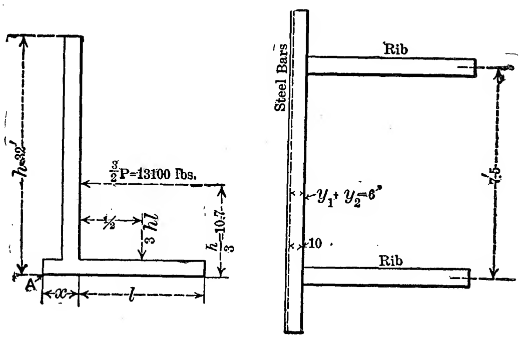


FIG. 107.

$=32$ ft.; angle of repose of earth $=36^\circ$ $18'$. Then $ACD=90^\circ-36^\circ$ $18'=53^\circ$ $42'$, $\frac{1}{2}ACD=26^\circ$ $51'$; and tang 26° $51'=0.5060$ (see also page 61 of this volume).

Substituting the above values in Coulomb's formula, namely,

$$P=\tfrac{1}{2}wh^2 \tan^2 \tfrac{1}{2}ACD,$$

we have

$$P=13{,}100 \text{ lbs.}, \quad \tfrac{3}{2}P=19{,}600 \text{ lbs.}, \quad \frac{P}{2}=6500 \text{ lbs.}$$

This resultant pressure P is assumed to act horizontally on the front face at a point $\frac{1}{3}h$ from the bottom. The tendency of the front face to overturn is balanced by the earth load on the base-slab. Let l be the length and x the extension of this slab to the front. The weight of earth on the slab per foot of length of wall is whl; hence equality of moments about A gives

$$\tfrac{3}{2}P\frac{h}{3}=whl\left(\frac{l}{2}+x\right).$$

Substituting values of P, w, h, and assuming $x=3$ ft., we find $l=9.4$ ft. nearly. In the actual design $l=10$ ft.

Mr. Graff in his discussion assumes that the total pressure P as found by formula is double what it should be, and assumes $P=\frac{13,100}{2}=6500$ lbs. To find the intensity p on a unit area of the base of the wall,

$$\frac{p}{2}\times 30=6500; \therefore p=433 \text{ lbs.}$$

It may be noted that Mr. Rankine's formulæ give $P=12,500$ lbs., and the intensity at bottom of wall, p, $=800$ lbs. Taking a height of one foot of face wall 7.5 ft. in length, the distance between ribs, and considering this a beam fixed at both ends, and independent of the remaining facing of the wall, we find the bending moment at centre of this beam to be

$$M=\frac{1}{12}\frac{wl}{12}=\frac{1}{12}\times\frac{433}{12}\times(7.5\times 12)^2=24,350 \text{ in.-lbs.}$$

nearly. And allowing a factor of safety of 4,

$$M=97,400 \text{ in.-lbs.}$$

Using Johnson's formulæ or diagrams (see Catalogue of the Expanded Metal Fireproofing Co., St. Louis), Mr. Graff finds $y_1+y_2=6$ ins., $\frac{a^2b}{d}=0.5$ sq. ins., or $\frac{3}{4}$-in. rods spaced 9 ins. gives excess of strength. Adding 4 ins. outside of metal, $h=6+4=10$ ins. In these expressions b = width of section in inches; a^2 = area of one bar in square inches; d = spacing of bars in inches; $\frac{a^2}{d}$ = number of square inches of metal per inch of width; $\frac{a^2b}{d}$ = total quantity of metal in width b; e = distance in inches from extreme fiber on tension side to middle plane of metal reinforcement;

and $h = y_1 + y_2 + e$, in which y_1 is the distance of the extreme fibre in compression, and y_2 is the distance from the neutral axis to the middle plane of the metal reinforcement. With 1:3:6 rock concrete, $E_c = 3{,}000{,}000$, $f_c = 2000$, $f_t = 200$, $E_s = 29{,}000{,}000$, and the limit of elasticity of steel $F = 50{,}000$ lbs. per square inch. The above quantities are related as follows: $y_2 = 1.72 y_1$, $\frac{a^2 b}{d} = 0.0195 b y_1$, $M_0 = 2750 b y_1^2$.

Further computations, not necessary to be given in this volume, may be found in *Eng. News*, March 9, 1905.

Quantities for the plain-concrete wall and the reinforced wall are given in the following table. In this the concrete per linear foot of walls is given. In the reinforced wall the steel is evaluated in terms of concrete at $6 per cubic yard in place. The steel is figured at $4\frac{1}{2}$ cents per pound in place.

Height of Wall.	Concrete in Plain Wall, Cu. Ft.	Reinforced Wall, Cu. Ft.	Saving, Per Cent.
40	396.4	218	45
30	226.0	127.8	43.3
20	110.0	69 9	36.4
10	44.0	34.9	20.4

The author does not think the above is altogether a fair comparison. Theory calls for a wall only $\frac{1}{3}$ of the height, and not $\frac{4}{10}$ of the height. The weight of the earth on the steps at the back of the wall is never considered in determining the stability of the wall. If the wall is built hollow and the space filled with broken stone, sand, or compacted earth, sufficient stability will be insured. Few engineers, however, are willing to risk anything less than a solid wall of $\frac{1}{3}$ its height. The author, however, fails to see that the reinforcement adds anything to the stability of a wall against overturning bodily as a whole. It will undoubtedly prevent the wall giving way by local displacement of its parts, the usual way in which walls give way. As a matter of fact little is definitely known as to the amount of pressure and its distribution against a retaining-wall.

Stiff earths frequently stand for a long time in any climate without any support, and then yield only gradually and a little at a time: there is never any sliding of a definite mass along a theoretical plane of rupture. Many excavations and tunnels are held to the vertical or a given slope by a thin plastering on the exposed face.

The elaborate computations to determine the number, size, and distribution of reinforcing-rods in concrete retaining-walls would seem unnecessary. A series of rods placed by some rule of thumb would give doubtless as good results, just as the common practice of making unreinforced-concrete walls $\frac{4}{10}$ of the height. There is, however, some satisfaction, when a definite and reasonable theory is well established, in following the theory with a good margin of safety added. We need only consider, therefore, whether there is a well-established and reasonable working theory by which the amount and distribution of reinforcing-rods can be determined. Mr. Johnson says: "Tests made by M. Considère and supplemented by a similar set of experiments made by Prof. W. K. Hatt have shown that concrete when reinforced by steel will distort without cracking from ten to twenty times as much as it would be capable of without the metal reinforcement. Were it not for this surprising quality of ductility of the concrete when armed, steel-concrete constructions would be a failure." Recent experiments by Messrs. Turneaure and Talbot indicate that no such valuable and all-important property is imparted to concrete by metal reinforcement. The conservative engineer must therefore await the result of experience before blindly giving up the old and tried rule of $\frac{1}{3}$ or $\frac{4}{10}$ of the height for the thickness of retaining-walls. Much has unquestionably been accomplished by metal reinforcement in beams, floor-slabs, and arches, resulting in better, stronger, and more economical constructions. These have been fully tested by experience and experiment. And it is to be hoped that similar results may follow the use of reinforced-concrete retaining-walls.

In *Engineering News*, Jan. 5, 1905, are given drawings of plain-concrete and reinforced-concrete retaining-walls. The Howe formulæ were used in determining the resultant earth-pressure and the angle it makes with the horizon, namely:

$$E=\frac{H^2y}{2}\sqrt{\tan^2\alpha+\tan^4\left(45^\circ-\frac{\phi}{2}\right)},$$

$$\tan d=\frac{\tan\alpha}{\tan^2\left(45^\circ-\frac{\phi}{2}\right)}.$$

H = vertical height of wall plus the surcharge;
W = weight of a cubic foot of concrete = 150 lbs.;
y = " " " " " earth = 100 "
ϕ = angle of repose of earth = 30°;
α = " back of wall makes with the vertical;
E = resultant earth-pressure in pounds per linear foot of wall;
d = angle resultant E makes with the horizontal.

The A. L. Johnson formula was used for the reinforced concrete. The working unit stress for steel of 50,000 lbs. per square inch elastic limit was 12,500 lbs. per square inch, and for steel of 32,000 lbs. per square inch it was 8000 lbs. The working stress for imbedment of the bars is assumed to be 250 lbs. per linear inch, and for plain bars 160 lbs. For the special case given there is \$1.15 per linear foot in favor of the reinforced-concrete wall of equal stability with the plain-concrete wall.

CONCRETE-STEEL PILES.

THERE are several types. It will be noticed that those described below are rectilinear in cross-section and with one exception are quadrilateral. In Fig. 108 is shown a French form The top is shouldered in with a round section, for the application of the driving-cap, and the point is pyramidal. The point is fitted with a cast-iron shoe consisting of a point and four straps, which bend inwards, locking into the concrete.

The reinforcement consists of four round soft-iron rods placed at the corners and wired together. In some instances a hole is left in the centre line and bifurcating to the sides near the point; this is for the purpose of using the water-jet in driving the pile, or for forcing a cement grout into the surrounding soil. For sheet-piles the section is rectangular and has an additional rod at the middle of the long faces. A wedge-shape edge is formed

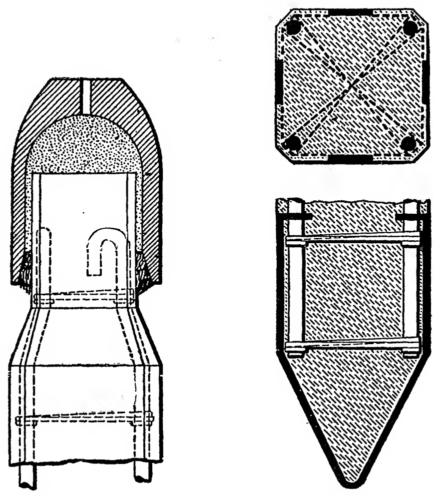


FIG. 108.—REINFORCED-CONCRETE PILE OF HENNEBIQUE CONSTRUCTION.

at bottom, and semicircular grooves are formed in the narrow faces. These circular holes are for the application of the water-jet in driving or forcing concrete into the surrounding soil, and are finally filled with grout so as to form a monolithic wall of sheeting (see Fig. 108).

Another type of sheet-piles is shown in Fig. 109. The reinforcement consists of two eye-beams, tied together at intervals by means of plates and angles. One of the narrow faces has a groove and the other a tongue in order not to have any open joints after driving. The pile shown was used for a quay wall, and at the top is shown a part of a land-tie, or anchor rod, for the purpose of tying the wall to a massive concrete wall.

Fig. 110 illustrates a form of concrete-steel pile employed in constructing a wharf in Russia. These piles were $15\frac{3}{4}$ inches square, pointed, and reinforced as shown in the drawings. The reinforcement weighed 45 lbs. per linear foot of pile, and the total weight per linear foot was 262 lbs. Each pile supported 269 sq. ft. of wharf area, carrying a load of 85,584 lbs.

An American pile is shown in Fig. 111. The pile is square in cross-section with pyramidal point and flat top. The reinforcement consists of four $1\frac{7}{16}$-in. rods placed at corners and extending down into the pyramidal point, being tied together at

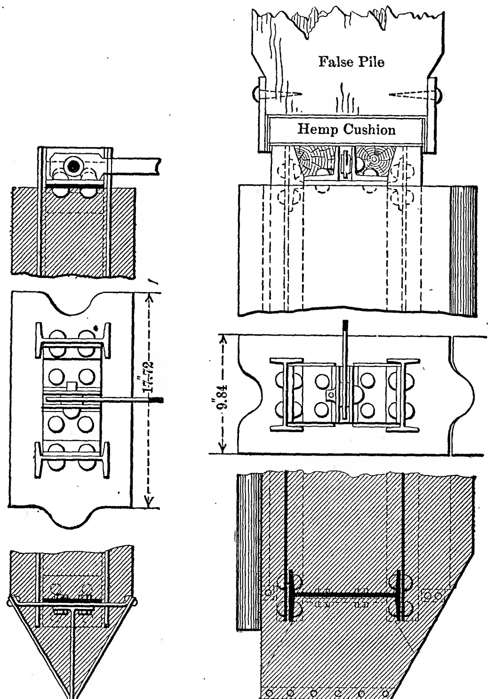


FIG. 109.—REINFORCED-CONCRETE SHEET-PILE WITH I-BEAM SKELETON.

intervals with iron wire. In moulding these piles the concrete is carried 2 ins. above the rods. After the piles are driven this may be broken off, leaving the ends of the rods free, which admits of bonding the superstructure to the top of the pile.

The kinds of concrete piles described are moulded complete in proper forms, and after hardening are driven in the same manner as are timber piles.

Concrete-steel Piles Built in Place.—In Fig. 112 is shown a type of concrete-steel pile built in place, known as the "Simplex" concrete pile. This pile is reinforced by a circumferential cylinder of expanded metal, having a thickness of $\frac{5}{16}$ inch and a 3-in. mesh. Fig. 113 illustrates the Raymond concrete pile. The reinforcement consists of a round bar, $1\frac{1}{2}$ ins. diameter, extending along the axis of the pile, and of three similar bars distributed along the circumference; these bars are of $\frac{3}{4}$-in. diameter.

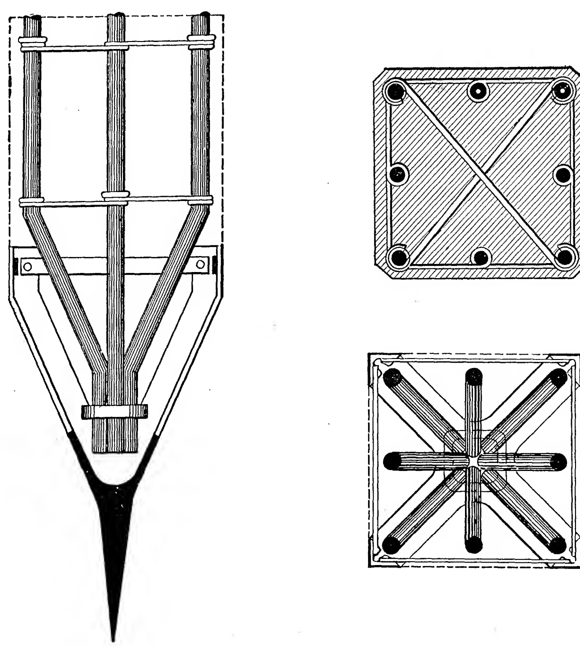


FIG. 110.—REINFORCED-CONCRETE PILE FOR WHARF AT NOVOROSSISK, RUSSIA.

Concrete Pile Foundations.—In this country as well as in Europe there are numbers of pile foundations for important structures, such as large buildings and bridges. The piles may be of any convenient size, 12 to 18 ins. square. After moulding and when sufficient time to harden has elapsed, the piles are driven in the usual manner. Commonly hammers weighing from 5000 to 9000 lbs. are employed. The height of fall is from 1 to 5 ft.

It is necessary to place a cap of some kind for a cushion, which receives the blow and prevents the crushing of the concrete; or the water-jet may be employed either separately or in conjunction with the driving. The cap may be of cast steel. In Fig. 108 is shown a cup-shaped casting which fits loosely the neck of the pile. The annular opening is calked at its lower edge with clay and tow, and the hollow space above is filled with sand or sawdust through a hole in the top of the cap. Other devices are employed, such as a socket-cap, where the space imme-

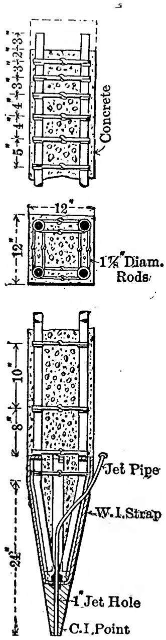


FIG. 111.

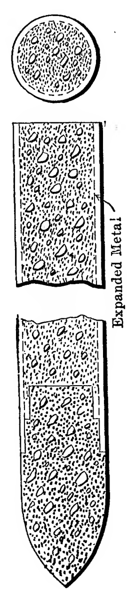


FIG. 112.

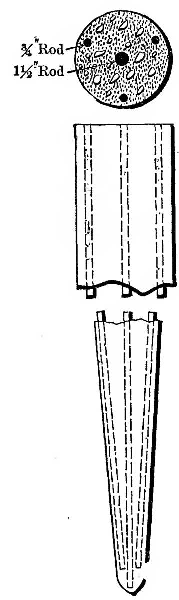


FIG. 113.

FIG. 111.—REINFORCED-CONCRETE PILE FOR BUILDING FOUNDATION AT NEW YORK CITY.
FIG. 112.—"SIMPLEX" PILE OF REINFORCED CONCRETE BUILT IN PLACE.
FIG. 113.—"RAYMOND" PILE OF REINFORCED CONCRETE BUILT IN PLACE.

diately above the pile is filled with sawdust and the upper socket receives a hard wood block; again, a cap of alternate layers of iron plate, wood, and lead. In sinking concrete piles the water-jet is the safest method to employ.

Arrangement of Piles in Foundations.—The spacing of piles in foundations is similar in every respect to that for wooden piles. Commonly the piles are spaced farther apart, and singly or in clusters. The floor or platform resting on the piles may be constructed with iron beams resting on piles, and the open spaces filled with concrete, or reinforced-concrete slabs may be constructed on the piles. Under a bridge pier in France the foundation consists of eleven piles of the Hennebique form (see Fig. 108). The piles are 16 ft. in length and 16 ins. square. They were driven through $6\frac{1}{2}$ ft. of mud and into the underlying gravel. Under a hammer weighing 8800 lbs., with a drop of 20 ins., the piles penetrated 8 to 12 ins. at each blow into the soft material; in the gravel with a drop of $2\frac{1}{3}$ to $4\frac{1}{4}$ ft. the penetration was 2 ins. and gradually decreasing to 0.4 in. The ultimate bearing power of these piles computed from Ritter's formula is as follows:

$$W=\frac{h}{e}\frac{Q^2}{Q+q}+Q+q=546{,}000 \text{ lbs. for each pile;}$$

where
Q = weight of hammer in pounds = 8800
q = " " pile in pounds = 8800
h = height of fall in inches = 48
e = final penetration in inches = 0.4

Under the Hallenbeck Building, ten stories in height, the American type of pile (see Fig. 111) was employed. The piles were driven in clusters of two, four, and six piles, spaced 12 to 18 ft., and capped with reinforced-concrete slabs, with four eye-beams upon which the supporting columns rested directly (see Figs. 114 and 115). The piles were intended to carry 80,000 lbs. each, 36,000 lbs. per square foot allowed under the New York Building Laws, and 44,000 borne by the four reinforcing-rods.

Sheet-piles.—In driving sheet-piles 11.8×14.8 ins. in France a hammer weighing 3520 lbs., with a $6\frac{1}{4}$-ft. drop, was employed. And in Russia sheet-piles were driven 2 ft. into hard bottom with from 10 to 12 blows of a 3375-lb. hammer falling 12 to 14 ft. Thus the methods of driving hardened concrete piles are entirely similar to those used in driving wooden piles.

Constructing Piles in Place. — The phrase "Piles constructed in place" simply means that a pointed metal cylinder or casing is first driven into the ground to the required depth

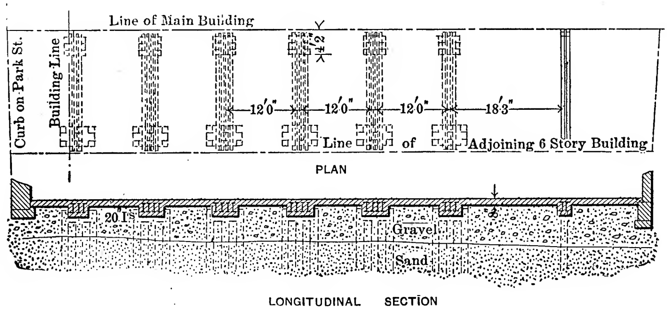


FIG. 114.—REINFORCED-CONCRETE PILE FOUNDATION FOR HALLENBECK BUILDING, NEW YORK CITY.

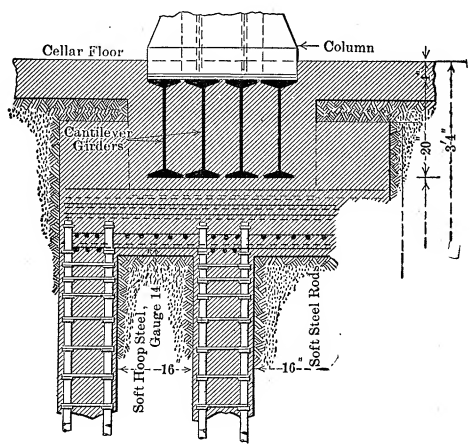


FIG. 115.—SECTION SHOWING REINFORCED-CONCRETE PILES AND PILE CAPPING FOR HALLENBECK BUILDING FOUNDATIONS.

and then filled with concrete, either with or without iron reinforcement. In Fig. 116 is shown the method adopted in constructing the simplex pile. A wrought-iron cylinder is fitted to a previously cast concrete point. This cylinder with its point is driven by means of an ordinary pile- driver to the depth required; the blow of the hammer is received by an oak block or cap. After driving, a cylinder of expanded-metal reinforcement is lowered in the iron cylinder until it rests on the concrete point. Concrete is then shovelled into the cylinder, the driving or outside cylinder being gradually raised as the filling progresses. A new point is fitted to the driving-cylinder and the same operation continued for each pile.

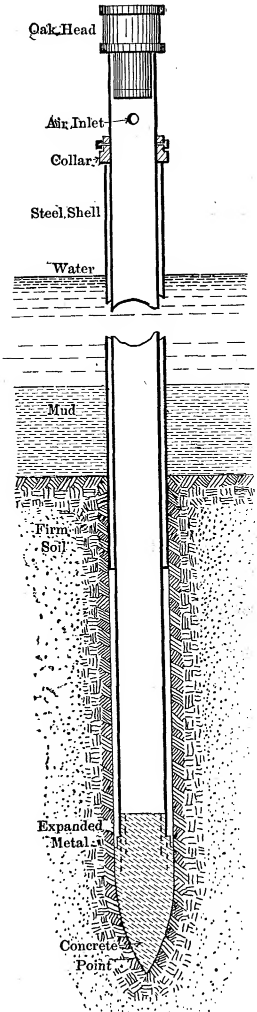


FIG. 116.—SIMPLEX DRIVING-SHELL FOR MOULDING REINFORCED-CONCRETE PILES IN PLACE.

In driving in water, a short outside cylinder is clamped and carried down with the driving-cylinder. The short cylinder is of sufficient length to reach well into the bed of the stream and remains in place. After loosening the clamp the driving-cylinder is withdrawn as before described.

The Simplex pile was used in constructing the foundations of the Engineer School at Washington Barracks. The piles were driven through yellow clay, blue clay, or silt somewhat compact, and into wet sand and gravel. As the upper strata were alternately wet and dry, wooden piles could not be used.

For concrete piles the apparatus consisted of a wrought-iron shell, an interior driving-form, a concrete point, a pile-driver, and frame and tackle for drawing. The wrought-iron shells were of boiler-steel, $\frac{3}{8}$ in. thick and 11 ft. long. The interior driving-form was a 10-in. cast-iron pipe; this was fitted to the concrete point, and a cushion-block of black ash was placed at the top to receive the blow. With a fall of 10 ft. and weight of hammer of 2200 lbs. the final penetration at the last blows was about $\frac{1}{3}$ in. in sand. In order to reduce the resistance to the withdrawing of the driving-pipe it was greased or oiled on the outside. It generally brought up with it the upper part of the concrete point, which was broken in driving. This was not of serious moment. When the point had reached the proper depth the concrete was placed in layers of about 12 ins., the driving-pile being raised so that its lower edge was about 6 ins. below the top of the concrete; this formed a plug in the bottom of the pipe and prevented inflow of water. The general length of the piles was 13 ft., length of point 3 ft. Some of the piles were 35 ft. in length and 17 ins. in diameter. The concrete points were composed of 1 part vulcanite Portland cement, 2 parts concrete sand, and 5 parts broken stone (run of the crusher), all by volume. The concrete of the pile was 1 part Old Dominion Portland cement, 3 parts concrete sand, and 5 parts gravel by volume. In order to determine the bearing power of these piles, one pile after completion was tested and dug up. Before the first test it was allowed to stand for two weeks. During a period of ten days the testing load was increased from 578 to 41,095 lbs. without the slightest evidence of settlement. When dug up, the pile showed a uniform cross-section and the point was intact, but not very well bonded to the upper portion of the pile. The pile rested, however, squarely on the point and therefore had a good bearing.

The Raymond Concrete-steel Pile.—This is another form of pile constructed in place. In the Raymond pile construction there is an iron driving-core made up of two semicylinders or cones, these having two or more semi-wedged-shaped recesses facing each other (see Figs. 113 and 117). Between these semicylinders are a number of iron wedges connected with a long bar. When this is pressed downward the semicylinders separate to a certain extent, and when the wedges are lifted they come together, or, as generally expressed, the cone collapses and can readily be withdrawn. The head of this cone has a driving-cap attached. A number of shells of steel plate are prepared and kept ready for use. These are made in sections and telescope into each other for ease of handling and convenience of storing. The driving-cone having been set in the leads of the driver, the wedges are pushed downward so as to spread the two halves of the driving-cone; an outer casing is now placed under the cone and slipped upwards until

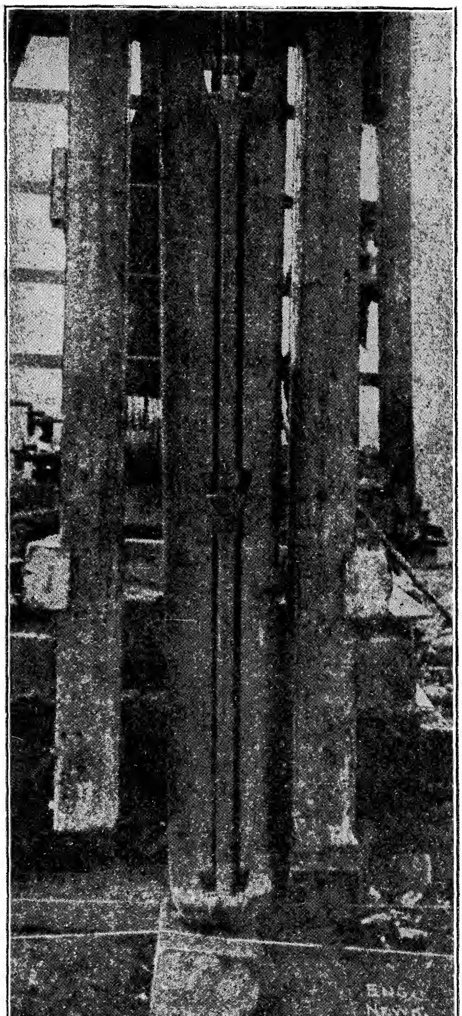


FIG. 117.—"RAYMOND" DRIVING-SHELL CORE EXPANDED READY FOR DRIVING.

the driving-cone is encased in the outer shell. The core with its shell is now driven into the ground to the proper depth. The wedges lifted, the driving-cone collapses and is easily withdrawn, leaving the outer shell in place. The reinforcing-bars (if used) are now inserted in the shell and the shell is filled with concrete. In constructing a building at Aurora, Ill., 142 piles 20 ins. diameter at top and 13 ins. at bottom were driven by means of an ordinary pile-driver with a 2400-lb. hammer falling 20 to 25 ft. The penetration at each blow ranged from $\frac{1}{4}$ in. to 2 ins.

The shell used was made of No. 20 sheet iron, and the concrete was composed of 1 part Portland cement, 2 parts sand, and 4 parts broken stone. The material penetrated was filled ground overlying rock at a depth of 14 ft.

The only advantage to be claimed for concrete piles is that of durability: it will not rot. It is frequently stated that a concrete pile will carry a greater load per pile than a wooden pile Why this is so is not clear. When moulded before driving they cannot be subjected to much hammering without serious damage; they must therefore be of comparatively short length and driven in a soft or pliable material. When formed by filling a small tube or casing, it is not by any means certain whether we secure a homogeneous and monolithic column or not, and it may be little better than if the casing is filled with sand or broken stone.

CONCRETE-PILE FOUNDATION.

CONCRETE piles were driven for the foundations of the U. S. Express Company's building, New York City.

The permanent moisture-line below the surface is about 10 ft. Had wooden piles been used, a trench or excavation to that depth would have been necessary in order that the top of piles should be under a condition of constant moisture. The bearing power of concrete piles is greater than that of wooden piles, as they have a diameter of 6 ins. at small end and of from 18 to 20 ins. at large end. It also seems to be claimed that the driving of the iron core does not cause the same danger to adjacent buildings and consequently does not require underpinning or a resort to other means of protection. Exactly why this is the case does not seem clear. In the present case the Raymond system was adopted, and there were required 160 piles of about 25 ft. in length. A test pile was driven to a depth of $25\frac{3}{4}$ ft. with a 3100-lb. hammer. The penetration was $\frac{1}{2}$ in. at last blow, the hammer falling 8 ft. The core was withdrawn, the shell filled with concrete, and a platform placed on top and loaded with 25 tons of pig iron laid gradually. With this weight the settlement was $\frac{1}{16}$ in. Increasing the load, a total settlement of $\frac{9}{32}$ in. was observed. The final load was 38 tons. When this load had remained on for a period of 24 hours, the settlement increased to $\frac{13}{32}$ in., and 24 hours later to $\frac{15}{32}$ in., and no further increase was observed. The *Engineering News* formula gives for such a pile a safe load of $16\frac{1}{2}$ tons. The number of piles driven in ten hours was about fourteen. The pile-driver crew consisted of eight men, and the concreting gang of six men. A concrete pile 25 ft. long contains about 0.8 cu. yd. of concrete composed of 1 cement, 2 sand, and 4 broken stone.

The steel shell for each pile is made in four sections of No. 20 steel, the sections telescoping one within the other for convenience in handling and storing. When the core is pulled and lifted high enough to place one of the telescoped shells, the shell is pulled up, covering the core. The cap and oak striking-block are placed on top, when the pile is ready for driving. After driving to the proper depth the core is "collapsed" and withdrawn, and the shell filled with concrete. A solid concrete pile thus formed is excellent, and no objection can be made to the use of concrete piles except the cost, which is about \$1.15 per linear foot, varying, of course, with the cost of material and labor. But what assurance can we have that the filling of a long tube with small diameter is or can be well executed? For separation of the stone and mortar is almost certain to occur by whatever process the shell is filled. From the author's observations on any method of depositing concrete by dropping from a height, such a column is little better than filling the shell with sand or broken stone. The building above referred to has no basement; the ground-floor is paved with asphalt blocks over which the heavily loaded wagons will be driven.

The foregoing description (but not the opinion expressed) is taken from *Eng. News*, Oct. 20, 1904. (See also *Eng. News*, June 20, 1901; Dec. 11, 1902; March 4, 1904.)

As a rule Portland cement should be used in making reinforced concrete.

Excellence of materials and manufacture is quite as important an element in producing a serviceable and durable concrete-steel structure as is excellence in design. In fact, a poor design is more safely allowable for such a structure than are poor materials and workmanship. In receiving this caution against poor materials and workmanship, it should not be misinterpreted to mean that impracticable requirements of manufacture are necessary for good results. This is far from the actual fact. Only the ordinary standards of excellence are demanded. In fact, in insisting upon the need of a high-quality mixture for reinforced concrete work, care should be taken not to carry the demand to extremes. To demand for a floor filling a concrete equal in quality to that in the reinforced plate, or for a bridge abutment a mixture as rich as that in the arch ring, is usually unnecessary for the stability of the structure, and is poor engineering. This is a structural principle which has been well worked out in the practice of European engineers of concrete-steel work. As an example, in building the Melan arch at Laibach, Austria, described in *Engineering News* of July 16, 1903, the heavy abutments were begun with a 1:14 concrete, and the mixture was gradually increased in richness to a 1:3:5 concrete at the top.

The matter of the proper composition of concrete for concrete-steel naturally follows from what has just been said. Practice furnishes a wide range of mixtures from which to choose. In Monier construction a mortar composed of one part cement and three parts sand is used for thin slabs for floors, partitions, tanks, conduits, etc., and a mortar composed of one part cement to 4 or $4\frac{1}{2}$ parts of sand is used for arch bridges and similar heavy work. Some others of the European concrete-steel constructions use a mixture of cement and sand only, but the greater number employ a true concrete of cement, sand, and broken stone. In the United States concrete is employed almost exclusively. The proportions adopted are 1:2:3, 1:1½:4, 1:2:4, and 1:3:5. The most common mixtures are 1:2:4 and 1:3:5, but there is no uniformity of practice and apparently no settled opinion as to the best mixture for general use or for special classes of construction.

There is a tendency at present to use leaner mixtures for concrete-steel work than have been used in the past. This is based largely on the fact that lean mixtures are less likely to develop shrinkage cracks and fissures, and are stronger, provided the mortar is sufficient in quantity to fill thoroughly the voids. The danger from using leaner mixtures lies in the fact that a more careful grading of the aggregates is necessary and that the mixing must be unusually thorough, and these are the things which are the most difficult to secure in ordinary work. European engineers have made a success of lean mixtures for the same reason that they succeed with dry mixtures; they put a large amount of work into the manufacture and deposition. A rich mixture, like a wet mixture, assures safety against insufficient and inefficient labor in mixing and ramming the concrete. European builders in reinforced concrete are as a rule advocates of lean mixtures and dry mixtures, but they recognize their permissibility only in case the mixing, deposition, and compacting are performed with exceeding care.

In the United States wet mixtures are now almost universally employed for concrete-steel construction. This choice is dictated largely by practical considerations. With the amount of care which it is ordinarily practicable to secure, a more dense and homogeneous concrete is obtained with a wet than with a dry mixture. It is also practically necessary, in order to insure the thorough embedding of the reinforcement, that the concrete be wet enough to flow readily under moderate ramming into all the interstices of the metal skeleton. This quality is particularly desirable when the reinforcing elements are of small size and are closely spaced. The adhesive strength of concrete to embedded steel is largely effected by the continuity of the bond between the steel and concrete, and continuity of bond or perfect contact are most easily obtained with wet mixtures. Perfect contact is also necessary, as has been shown by recent tests, for the preservation of the embedded steel from corrosion. Another decided advantage of wet concrete is that a more impermeable mixture is secured.

These practical advantages of wet mixtures for reinforced concrete are held by experienced engineers generally to be sufficient to overbalance such objections to them as actually or theoretically exist. In this connection it must be remembered that the fear once anticipated, that a serious loss of strength would result from the use of an excess of water, has been shown to be largely unwarranted. The most complete and trustworthy experiments to determine the relative strengths of wet and dry mixtures are those made by Mr. Geo. W. Rafter, M. Am. Soc. C. E., in 1896, for the State Engineer's office of New York. Mr. Rafter made 12-in. cubes of dry, plastic, and wet mixtures. In the dry blocks the mortar was only a little more moist than damp earth; in the plastic blocks the mortar was of the ordinary consistency used by masons; in the wet blocks the water was so far in excess that the concrete quaked like liver under moderate ramming. The resulting average compressive strengths of the three sets of blocks were as follows:

Dry mixture..............	156 blocks,	2470 lbs. per sq. in.
Plastic mixture............	144 "	2294 " " " "
Wet mixture..............	148 "	2180 " " " "

The blocks tested were all from 18 to 24 months old. These tests, and in fact laboratory tests in general, indicate a somewhat greater strength for well-rammed dry mixtures. Evidence is not lacking, however, that wet mixtures develop greater strength ultimately. However this may be, the difference is not material. From a practical standpoint, the fact remains that unless there be a liberal use of water the mass cannot be properly consolidated by any reasonable amount of ramming. The use of a wet mixture economizes labor and produces a sounder and better product. In the case of concrete-steel there are the added advantages that a wet mixture insures a better bond between the concrete and the steel, and that it gives the steel better protection against corrosion.

REINFORCED-CONCRETE PILES WITH ENLARGED FOOTING.

IN Fig. 118 are shown concrete piles, elevation and section, used in underpinning the walls of a building. The

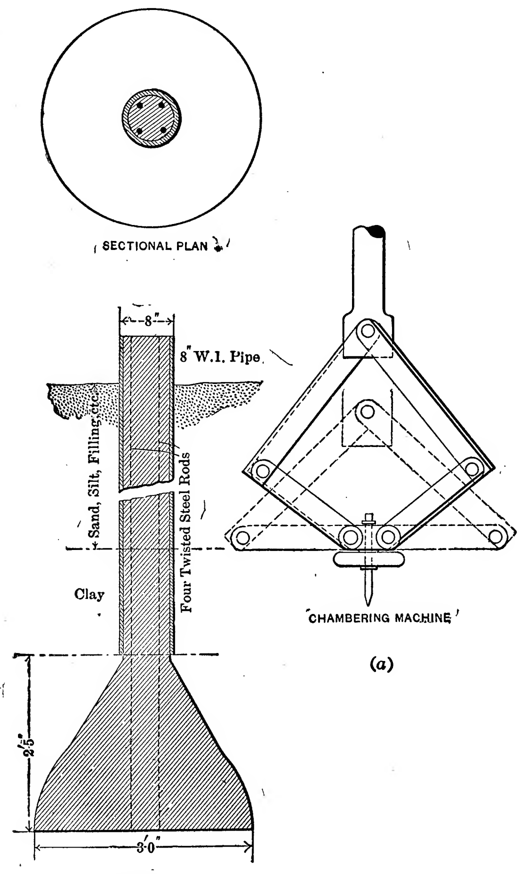


FIG. 118.—CONCRETE-STEEL PILE WITH ENLARGED FOOTING CONSTRUCTED AT BOSTON, MASS.

piles were placed in clusters under wall intersections and at column points, the load being carried to the piles by means of short eye-beams. The soft material overlying the stiff clay was about 20 ft. in depth. The total length of the pile is about 26 ft. The upper portion of the pile is 0.44 sq. ft. in section and has four twisted steel rods of square section embedded in the concrete. The 8-in. casing is first forced down through the soft material and into the clay to the depth of about 2 ft. Below this casing the clay was bored and washed out to an additional depth of 2½ ft. The casing had to be built up in short lengths of 5 ft. The apparatus for enlarging the footing below the casing is shown in Fig. 118 (*a*). Having washed out the loosened material, the whole interior was pumped dry, the steel rods inserted, and the concrete placed and rammed When the concrete reached the top of the enlarged portion, the casing was lifted a foot at a time and the concrete placed. The boring apparatus is made of angle-irons jointed. When closed it is run down the hole. Saw-teeth are formed on one leg of the angles, with a revolving motion, the material is cut up, and the enlarged footing formed. The area of the footing is about sixteen times that of the body of the pile, or 7 sq. ft. to 0.44 sq. ft. (See *Eng. News*, June 16, 1904.)

WATERPROOFING CONCRETE STRUCTURES.

CONCRETE structures are not impermeable, and some method of waterproofing is necessary for foundation walls, and the same may be said of arches, abutments, retaining-walls, and dams. One method of waterproofing is to cover the backs of walls with asphalt. First a coat of asphalt cut with naphtha is applied as a paint to the concrete when perfectly dry, and then this surface is covered with an asphaltic mastic composed of 1 part asphalt to 4 parts sand, this to be smoothed with hot irons and tamped and pressed in place. It is not difficult to obtain an asphalt which will not flow under a temperature of 212° F., and which will not become brittle when spread thin on glass at 15° F. below zero, and will resist the action of acids and alkalies. (For further description and requirements see *Eng. News*, Jan. 26, 1905.)

UNDERPINNING CONSTRUCTION.

Underpinning.—The construction of underpinning consists in removing in alternate sections a portion of the foundation and foundation-bed of a structure and substituting other supporting material, commonly providing a larger bearing area for the structure, for the purpose of reducing the load per square foot, where this has been too great for the underlying material. Or a portion or all of the foundation may be removed and the structure supported by direct supports until the new and permanent foundation is fully completed and built in close and full contact with the under side of the base of the structure.

Such work is attended always with more or less risk of causing damage to the structure. With intelligent plans and proper skill and care in the execution the largest and heaviest structure can be underpinned with safety.

The Washington Monument.—This monument is an obelisk-shaped tower. Its total height is 555 ft. and its base is 55 ft. square. It is constructed of white marble. The monument is for the main part a shell of masonry. When about completed to one-third of its height the pressure on its foundation-bed—a mixture of clay and sand—was about 5 tons per square foot (see page 347). It was decided to underpin the structure before completion. This was performed by tunnelling under the monument and enlarging the base with concrete. The superincumbent weight was about 15,000 tons. The work was executed without damage to the structure. This work was done to arrest further settling.

Columbus Monument.—The New York subway passes under this monument. It was therefore necessary to cut away a portion of its foundation, and to support the structure during the completion of this portion of the subway, and finally to arrange for a portion of the monument to rest on the roof of the completed subway. The original foundation was a mass of rubble masonry 45 ft. square and 12 ft. deep, resting on a bed of concrete 2 ft. in depth, the total depth of foundation being

14 ft. From the top of the rubble masonry to the top of the shaft is 75 ft. The base of the shaft is 35 ft. square. The weight of the monument, exclusive of the foundation masonry, is about 724 tons. The material on which that portion of the monument over the subway rested was gravel and sand. Shafts were sunk on the south and north sides of the foundation, and adjacent to the west line of the subway, to a depth of about 3 ft. below the bottom grade of the subway. From these shafts a tunnel 6 ft. wide and 7 ft. high was driven under the foundation of the monument, its east side corresponding with the west side of the subway, the bottom of the foundation forming the roof of the tunnel. The tunnel was filled with concrete to the level of the subgrade of the subway. A line of 12×12-in. yellow-pine columns, spaced 5 ft., was placed along the tunnel and wedged under the masonry of the foundation. The tunnel was then filled with rubble masonry and care taken to build the masonry solid to the foundation of the monument. That portion of the monument and its base outside of this new rubble wall was ultimately to be supported by the roof of the subway, and all that portion of the original rubble foundation in the way of the subway had to be removed. In order to do this some temporary support was necessarily to be provided. The excavation for the tunnel along the north, east, and south sides of the monument having been made to within 3 ft. of its sides, a channel, about 6 ft. in height and on a line about 4 ft. above the roof of the subway, was commenced in the masonry foundation—on the east side of the monument and running towards the west—and extended until the monument overhung about 5 ft. Four uprights 12×12 ins. were then set under the overhanging portion. The excavation was extended to the westerly line of the subway. On the completed concrete floor of the subway, north and south of the monument, framed trestles of timber 12×12 ins. were placed. Two 30-in. plate girders about 35 ft. in length were then skidded end on under the monument and resting on the framed bents of timber. Over the top cover-plates of the girders a 1-in. plank was placed, and all open spaces between girders and base of monument filled, in order to secure

even and uniform bearing. A 60-ton jack was used to lift the girders up against the base of the monument, and wedged by means of iron plates between the lower flange of the girders and the caps of the trestle.

The temporary timber posts and the old rubble under this portion of the monument were then removed, and earth excavated to grade of the subway, thus exposing the intended west wall of the subway. This was smoothed and plastered to receive the waterproofing which envelops the subway. The steel columns and beams of the subway were then placed in position. On the finished roof of the subway a rubble-masonry support was built under the overhanging portion of the monument and the roof of the subway, about 4 ft. in thickness. A course of wedging-stones was set about 3 ft. above the roof of the subway, and over these iron wedges were driven, thus relieving the 30-in. girders of a portion of their load. One girder was taken out, rubble built up and wedged as before, then the second girder removed and the space filled with rubble. All excavated spaces under the monument were then filled with masonry, leaving the monument supported in part on its old foundation and in part on the roof of the subway.

Columns of N. Y. Elevated Railway.—At Fifty-third Street and Broadway the double-track railway structure crossed the line of the subway, two of the columns of the elevated railway being directly over the centre row of subway columns, and placed between the two tracks of the street-surface railway. This elevated structure had to be supported during the construction of the subway. The following was the method adopted for supporting the south column. As it was intended to utilize the 30-in. by 35-ft. plate girders—the same as used under the Columbus monument—two trenches were excavated across Broadway and parallel to the elevated railway, each about 6 ft. wide and reaching to the bottom of the subway. One trench was midway between the two columns to be supported, and the other about 17 ft. outside of one column. A longitudinal trench joined these, being excavated between the two surface tracks. The permanent concrete of the floor of the subway was constructed in the transverse trenches and two framed trestle-bents placed in them, their tops being on a level with the bottom of the longitudinal trench. The plate girders were then placed on the trestle-bents and under the columns, requiring some cutting away of the masonry foundation of the columns. A grillage of 15-in. I beams was placed on the plate girders and bolted together with separators. These were floored over with 12×12-in. yellow-pine timbers, and upon these were erected trestle-bents reaching within about 4 ins. of the under side of the cross-girders of the elevated railway. These were thoroughly braced with 3×10-in. yellow-pine plank.

After removing the nuts of the anchor-bolts holding the cast-iron base to the foundation of brick masonry, the column with its cast-iron base was raised by means of jacks, and the load transferred to the temporary timber supports by means of wedges. The excavation around the pier and tearing out the foundation masonry of the columns was carried on and extended down to subgrade of the subway. The concrete floor and water-proofing were then put in place and the iron erected. The three subway columns supporting the column of the elevated structure were spaced 4 ft. 6 ins. centres, and the roof-beams increased from 18 to 20 ins. in depth. The arches of the roof were constructed and waterproofed. All that remained to be done was to build the masonry from the roof to the base of the column and let this down on it. Wooden forms for holding the concrete foundation were placed between the girders, the anchor-bolts with plate washers suspended in position, the bottoms of the rods being about 6 ins. above the waterproofing. Concrete was then placed as high up as the plate washers, and three 15-in. I beams 9 ft. 6 ins. long were placed in the concrete, the outside beams resting on the plates. The forms were then filled with concrete to about 2 ins. below the cast-iron base of the column and allowed to stand for four days. Grout was then forced between the concrete and the iron base. The column was loosened from the cross-girders and together with its base lowered about ½ in. into the grout to its original position. After eight days the main cross-girder of the railway was lowered on to the column and the timber framing removed. The other column was handled in the same manner, but separately. The dead load of the structure is given at 40,000 lbs., and the live load at 100,000 lbs., while the load on each of the three columns under the pier is about 120,000 lbs. (See *Eng. News*, Oct. 23, 1902.)

CONSTRUCTION ACROSS THE GREAT SALT LAKE, UTAH.

The purpose of this cut-off is to save a distance of about 45 miles and to lessen the rate of grade of the Southern Pacific Railway. In the main the construction is to be of a permanent character, constructed of shale and gravel dumped in place. The interesting portions of the work are the temporary tracks required to be constructed for the use of the construction trains. Over one stretch of about three miles there was found about 10 ft. of mud under the salt crust. A flooring of plank was laid, and upon this were placed sand bags filled with 100 lbs. of sand in three layers; upon the top layers a course of timbers 12×12 ins. ×10 ft. was placed, and on this three stringers 8×16 ins., upon which ties were placed and a temporary track laid. This temporary track was about 8 ft. south of the permanent track. Where water to the depth of 4 to 5 feet was encountered, temporary trestles were constructed. Each bent had four 40-ft. piles, capped with 12×12-in. timber, and spaced 15 ft. centres. Stringers were 8×16 ins. and 30 ft. long, on which ties and rails were laid. It is intended to fill in under and around these trestles with rock. The consistency of the bottom is very variable. In some cases a blow of the hammer would not drive a pile more than an inch or two, in others it would penetrate one or more feet at a blow. Sometimes a pile after having been driven from 30 to 50 ft. would rise several inches or feet after the hammer was lifted. (See page 217.)

In some places a single blow would drive a 26-ft. pile out of sight; a 28-ft. pile placed on top of the first was driven out of sight by another blow. Forty-foot piles were driven for a temporary trestle. Where permanent trestles are required piles 70 ft. in length will be used. There is about 12 miles of permanent trestle. This will be strengthened by using some ballast dumped around the piles.

Rubber boots used by the men were soon rendered useless by the action of salt water. (*Eng. News*, Nov. 27, 1902.)

FOUNDATIONS FOR THE NEW CAMBRIDGE BRIDGE, BOSTON, MASS.

THERE is nothing of special interest in the construction of these foundations except the record of pile-driving. Most of the structures in the vicinity of Boston rest upon timber piles. The machinery for driving the piles, cutting them off, and depositing the concrete was complete.

Some 15,000 piles were driven in the ten piers; these were brought from Maine. The piles are 40 or more feet in length. The heads of the piles are 18 ft., more or less, under water when driven the full depth.

The pile-driver is of the usual construction, with leads 75 ft. in length; it is fitted with a Warrington steam-hammer. The weight of the hammer and frame is 9800 lbs.; the striking part weighs 5000 lbs. The blow is due entirely to the fall of the hammer. The required steam pressure is 90 to 100 lbs., and 60 to 70 blows can be delivered per minute. The average day's work is over 100 piles driven in ten hours; as many as 212 piles have been driven in nine hours. A follower of oak 14 ins. square, capped with cast iron at both ends, and from 25 to 35 ft. in length, is employed when piles are driven below the water surface.

The piles are cut off below the water surface by an apparatus similar to the one described on pages 245 to 247 in this volume. The saw-shaft is 60 ft. in length, 4 ins. in diameter, but hollow. This slides up or down, so as to raise or lower the saw. The saw at the lower end of the shaft is 42 ins. in diameter and runs at a speed of from 400 to 500 revolutions per minute. As many as 600 to 800 piles, 10 ins. diameter, have been sawed off in ten hours.

In Fig. 119 is shown the steam-hammer. The hammer consists of a heavy ram on the end of the piston-rod which runs up through the frame into the steam-cylinder. The whole frame, including the hammer, is raised by an engine on the scow; the pile is then lifted and set in the leads; lowering the hammer on the pile causes it to sink through or into the mud; the steam, turned on, lifts and lets fall the hammer rapidly until the pile is driven to the required depth, when the steam is turned off.

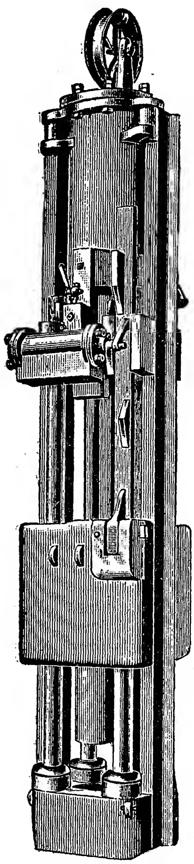


FIG. 119.—WARRINGTON STEAM PILE-DRIVER FOR PIERS OF NEW CAMBRIDGE BRIDGE, BOSTON, MASS.

The only purpose of the enclosing coffer-dam is to hold the concrete, and, after this is laid, to allow the water above to be pumped out so that the masonry may be constructed. No bracing is, therefore, required until the water is pumped out. The coffer-dam is constructed by driving a series of guide-piles about 10 ft. apart; these are braced by inclined piles. The piles are capped with heavy timbers, and three wales of 6×12-in. Southern pine are bolted on the inside. The sheeting consists of Southern pine 6 ins. thick and varying in width from 8 to 18 ins. It is driven by means of an ordinary driver. The edges are grooved for fillers $1\frac{1}{2}\times 3$ ins. The length of the pile is 40 ft.

The concrete filling is some 57,000 cu. yds. Lehigh Portland cement was used. The cement, sand, and gravel were placed in a box mixer and turned while the proper proportion of water was added. All of the ingredients were measured before being placed in the mixer. All the concrete was deposited under water through tubes having at top a diameter of 16 ins. and increasing to 22 at bottom. The tubes were kept full of concrete, which thus flowed out and did not have to drop from a height. By this means the ingredients did not separate, the stone and sand falling to the bottom and the cement collecting on top. Whether the pouring of the concrete throught tubes fully provides against this separation or not is a matter upon which engineers differ. This question has been discussed in another part of this volume (pages 102 and 103, also page 272). When the proper amount of concrete was laid the water was pumped, the braces inserted, and the masonry was commenced.

CENTRE-PIER FOR SWING-BRIDGE.

A 128-ft. wooden drawbridge over Barnegat Bay has been recently constructed. The draw-pier is cylindrical and 16 ft. diameter. Thirty-two piles 2 ft. 6 ins. centres were jetted down 12 ft. in a fine sand from 12 to 16 ft. in depth, underlaid with coarse sand and shells. These piles were cut off about 5 ft. above the bed of the bay. A steel caisson about 11 ft. high was placed around the piles. The shell of the caisson was composed of $\frac{1}{4}$-in. steel plates bolted together in sections, with an angle-iron stiffening-ring around the top. The interior of this cylinder was filled with concrete. The total weight of the structure is 160 tons, or 5 tons on each pile, assuming the piles to carry the whole load. Total cost of this structure $3759. The bids for a steel structure varied from $5500 to $22,000.

OVERDRIVING OF PILES.

THE great injury to wooden piles resulting from overdriving or hard driving is well known. Instances of this kind are mentioned in pages 210 and 352 of this volume.

It is not an overestimate to say that from thirty to fifty per cent of piles are seriously injured in driving. The points and heads are broomed and split. But the most serious injury will be found at a greater or less depth below the surface, which is rarely discovered or known, unless for some reason excavations are made adjacent to the piles, or they are pulled. This results from some such foolish rule as that a pile shall not penetrate more than $\frac{1}{8}$ to $\frac{1}{2}$ in. under blows of a hammer weighing 2000 lbs., more or less, and falling through a distance of 25 ft., more or less. Actual brooming, crushing, and splitting of piles are recorded as penetration, where there is no penetration.

In Fig. 120 is shown the usual manner in which piles are injured in driving. A 60-ft. pine pile was driven in soft mud with a Warrington steam-hammer weighing 5000 lbs. and falling

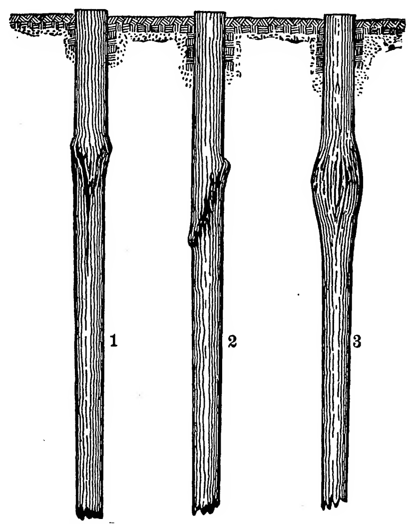


FIG. 120.—MODES OF FAILURE OF OVERDRIVEN PILES.

3 ft. It penetrated 42 ins. with eleven blows; at the last two blows it penetrated 7 ins. At the expiration of 38 hours the same pile, using the same hammer, penetrated only $6\frac{1}{2}$ ins. under eleven blows, $\frac{3}{4}$ in. under the first blow, and $\frac{1}{2}$ in. under the last.

A number of tests were made on piles which had been driven a month. In hard-pan the bearing had increased 30 to 50 per cent, while in soft material the increase was nearly 300 per cent. In the first case the length of the pile was 40 ft., and in the latter 70 to 75 ft. (*Eng. News*, June 9, 1904.)

For a more extended discussion of this subject see pages 207 to 219 of this volume.

It is clearly seen that the penetration at time of driving is not even suggestive of the ultimate bearing power of piles in soft materials. The length of the pile with its corresponding large surface will give ample bearing power in even very soft materials. The frictional resistance and necessary spreading of material increase directly with length of piles.

RAILWAY PILE-DRIVERS.

THE modern railway pile-driver is designed to be used for other purposes as well as driving piles, such as the erection of steel and iron bridges and also for wrecking-derricks. The following is a brief description of a type of driver and derrick designed by O. J. Travis, Illinois Central Railway.

The pile-driver leads range from 18 to 24 ft. in height. They are formed by riveting 4×6-in. angles back to back. (See Fig. 121.)

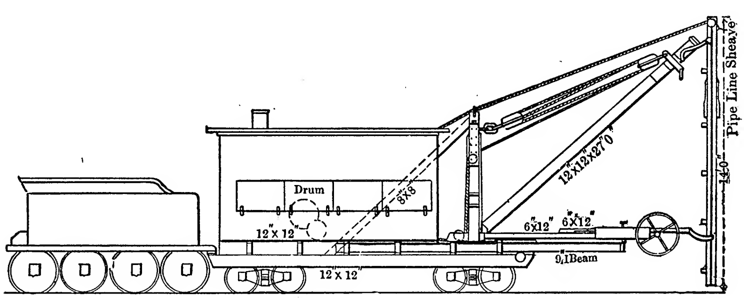


FIG. 121.—COMBINED PILE-DRIVER AND DERRICK CAR, ILL. CENT. R. R.

There is a heavy A frame with a boom 27 ft. in length, 12×12 in. section of timber. The pile-driver leads are hinged at the end of the boom. An extension strut can be run out in order to steady the leads at the bottom. There is a pair of 9-in. I beams placed in guides along the side sills of the car; these with a cross-beam at the front end can be run forward by means of a crank and cog-rack. This apparatus has been used in erecting truss spans up to 150 ft. in length without the use of a traveller. Steel plate girders of large pattern are handled with rapidity and safety. It has a lifting capacity of 30 tons. Other types are described and illustrated in *Eng. News*, Oct. 30, 1902.

The leads are as long as 45 ft., the weight of hammer usually from 3000 to 3500 lbs., and the ropes from $1\frac{1}{2}$ to 2 in. manila, or $\frac{7}{8}$-in. steel cable. In some cases the leads are of oak timber, 7×10 ins., reinforced with 7-in. channels, 17 lbs. per yard, leads spaced $22\frac{1}{2}$ ins. The leads can be raised or lowered in a minute or two. They are designed to drive piles at either end and on either side and in any reasonable position.

FORMS FOR MOULD-CLAMP.

THE usual practice of constructing forms or moulds for concrete is to set up vertical scantling in two rows, the proper interval apart; brace these strongly on the outside by inclined struts, then place a lining of rough or planed boards on the inside of both vertical frames and commence placing and ramming the concrete in the enclosed space. On long walls a low timber framing or box may be formed, 1 to 2 ft. in height, and the enclosed space filled with concrete. After the lower layer has set, this form is moved vertically upward and used again. In this method it will probably be necessary to hold the sides of the mould together by rods extending through the walls, which can be removed. These are slow and expensive methods. There are a number of devices in use, and where large quantities of concrete have to be laid continuously or at frequent intervals it becomes a matter of importance to have some well-designed frames ready for use at any time and anywhere. In Fig. 122 is shown a wall-mould clamp employed on the L. & N. R. R. by J. C. Van Natta (see *Eng. News*, Mar. 31, 1904). The clamp is made of malleable iron and is put through the form. The hooks are connected by a twisted wire. The cam is turned so as to produce the desired tension; the concrete is placed and rammed; loosening the cam, the eye-bolt is released from the wire tie; the form is removed and set up at some other position. The form may be framed any length. The wire is left

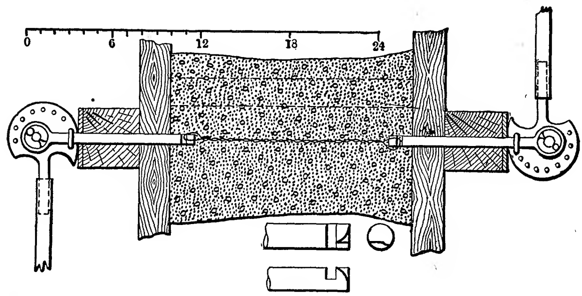


FIG. 122.—CLAMP AND TIE FOR WALL-MOULDS FOR CONCRETE WORK.

in the concrete and the bolt-hole filled. All piers and walls are reinforced on the L. & N. R. R. (*Eng. News*, March 31, 1904.)

WOODEN POLES.

THE poles for electric line work are commonly of wood, such as red and white cedar, cypress, juniper or redwood, and chestnut. While cedar has a comparatively long life, it is very scarce. It resists well the attacks of insects and fungus growths. This is, however, only true of the heart; the sap portion rots quickly at the ground surface.

Cypress will last from five to six years; heart-sawed pine poles, from eight to nine years; the sap will not last longer than three years.

In order to give durability, the butts of the poles have been charred to a point one foot above the ground, and then saturated with a coal-tar paint. Little or no benefit resulted. Creosoting seems to be the best and perhaps the only economical and practicable method of increasing the life of wooden poles. It costs about $20 per M, B.M., to creosote timber, and may increase the cost from two to three times that of the untreated timber. (See *Eng. News*, May 29, 1902.)

STRENGTH AND COST OF CONCRETE OF DIFFERENT COMPOSITIONS.

Number.	Proportions (cement = 1).		Yield in Terms of Volumes of Gravel and Broken Stone.	Age, Days.	Load on Cubes in Lbs. per Sq. In.		Average Crushing Strength, Lbs. per Sq. In.	Beams, 6×6×18 Ins.		Cost per Sq. Yd. for Depth of 6 ins., Cts.
	Gravel.	Broken Stone.			8 in.	6 in.		Load at Center, Lbs.	Mod. of Rupture, Lbs. per Sq. In.	
					Natural	Cement.				
1	3	3	.....	30	610	540	575	1,280	160	....
2	3	3	77%	90	870	470	670	1,270	159	49.0
					Portland	Cement.				
3	5	..	80.4%	30	1,750	1,565	1,657	2,700	338	77.5
4	5	..	.....	90	1,770	1,990	1,880	3,060	382	....
5	4	4	74%	30	1,500	1,365	1,432	2,440	305	74.0
6	4	4	.....	90	2,210	1,780	1,995	5,360	670	....
7	6	4	70.7%	30	1,180	1,170	1,175	1,800	225	68.6
8	6	4	.....	90	1,260	1,430	1,345	1,620	203	....
9	5	5	74.4%	30	2,250	1,545	1,897	2,800	350	66.1
10	5	5	.....	90	1,850	2,050	1,950	3,280	410	....
11	8	4	72.8%	30	890	840	865	1,650	206	57
12	8	4	.....	105	883	1,230	1,056	2,500	313	....
13	6	6	76%	30	1,290	895	1,092	2,550	319	64
14	6	6	.....	90	1,510	930	1,220	3,000	383	....
15	8	6	74%	30	965	570	767	1,750	219	56.8
16	8	6	.....	115	1,425	860	1,142	1,700	213	....
17	7	7	74%	30	1,240	1,365	1,302	2,100	263	56.6
18	7	7	.....	90	1,190	1,425	1,307	2,820	353	....
19	8	8	73%	30	1,010	990	1,000	1,600	200	51.4
20	8	8	.....	103	1,050	1,030	1,040	1,930	241	....
21	10	10	71.5%	30	530	600	565	1,010	126	50.3
22	10	10	.....	118	585	....	....	1,220	153	....
23	10	10	......	102	....	610	....	....	....	....
24	6*	6	.....	30	1,230	780†	1,230	1,780	223	....
25	6*	6	.....	107	1,720	1,535	1,627	2,410	301	....
26	8	16	70%	...	....	....	....	....	...	51

* Sand. † Defective; omitted from average.

TESTS OF PORTLAND CEMENT.

In the following the results of a number of tests made with Atlas Portland cement and a few with Utica cement, a natural cement, are given. The tests were made with 6-in. and 8-in. cubes and other pieces $6 \times 6 \times 21\frac{1}{2}$ ins.:

		Utica.	Atlas.
Passing sieve No. 50		97.4 p. c.	99.6 p. c.
" " " 100		80.5 "	92.5 "
Time from mixing to initial set		32 min.	1 hr. 56 min.
" " " " final "		3 hrs. 20 min.	5 hrs. 16 min.
Water		31 p. c.	17 p. c.
Age	24 hours	80 lbs. per sq. in.	265 lbs.
	7 days	181 " " " "	710 "
	30 "	230 " " " "	

The sand used was a fair quality of building sand, containing some clay and having about 35 p. c. of voids. Of the gravel 23 p. c. was retained on a No. 4 sieve, 53 p. c. on a No. 10, 71 p. c. on a No. 20, and 29 p. c. passed a No. 20. The broken stone was a good quality of stratified limestone crushed so that all of it passed a 2-in. screen and was retained on a $\frac{1}{2}$-in. screen (*Eng. News*, February 4, 1904.)

The table opposite gives strength and cost of concrete of different compositions.

From a number of experiments it is found that the coefficient of expansion of 1:2:4 Portland cement stone concrete is 0.0000055.

PLAN OF PILE SUPPORTS FOR THE PENNSYLVANIA R. R. TUNNEL UNDER NORTH RIVER.

Under certain portions of the tunnel screw-piles are employed to support the tunnel. This arrangement is shown in Fig. 121. (See *Eng. News*, Oct. 8 and 15, 1903.)

In *Eng. News*, Oct. 29, 1903, Mr. J. W. Reno suggested the use of wooden piles. The old plan calls for about 700 cast-iron screw-piles, 27 ins. diameter, spaced 15 ft. apart longitudinally, screwed down through holes in the floor of the tunnel

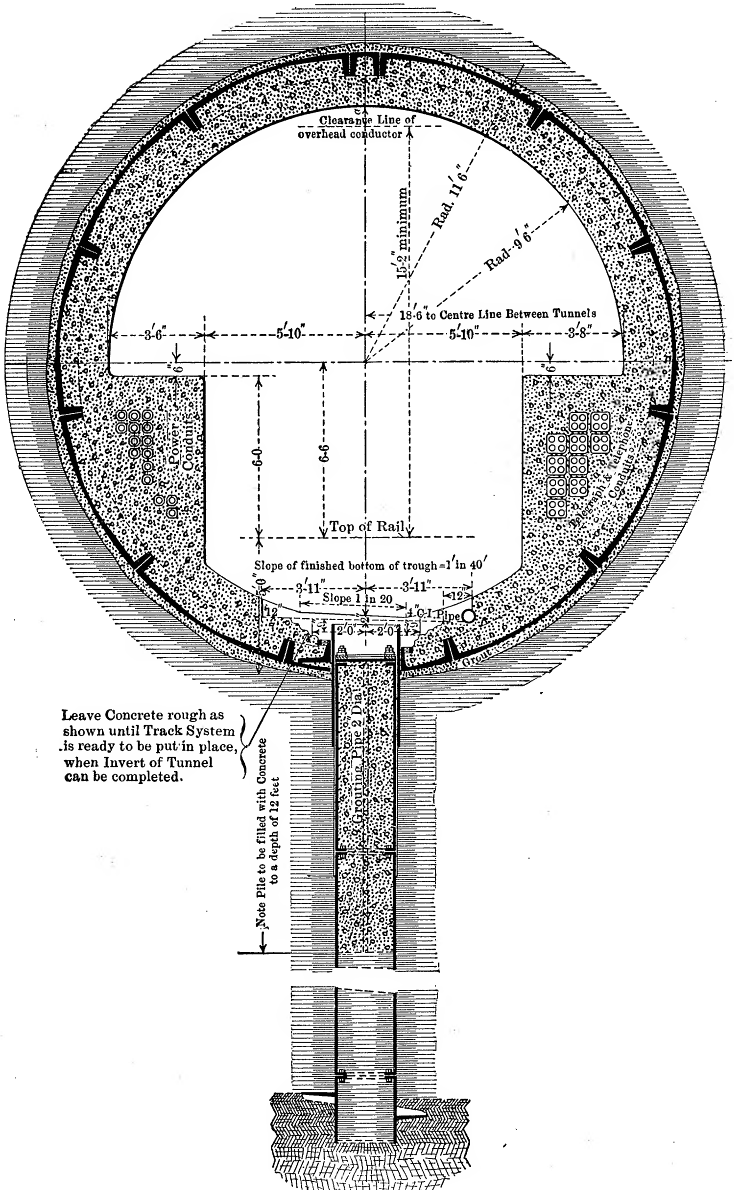


Fig. 123.—Typical Section of Subaqueous Tube Tunnel Supported by Screw-piles.

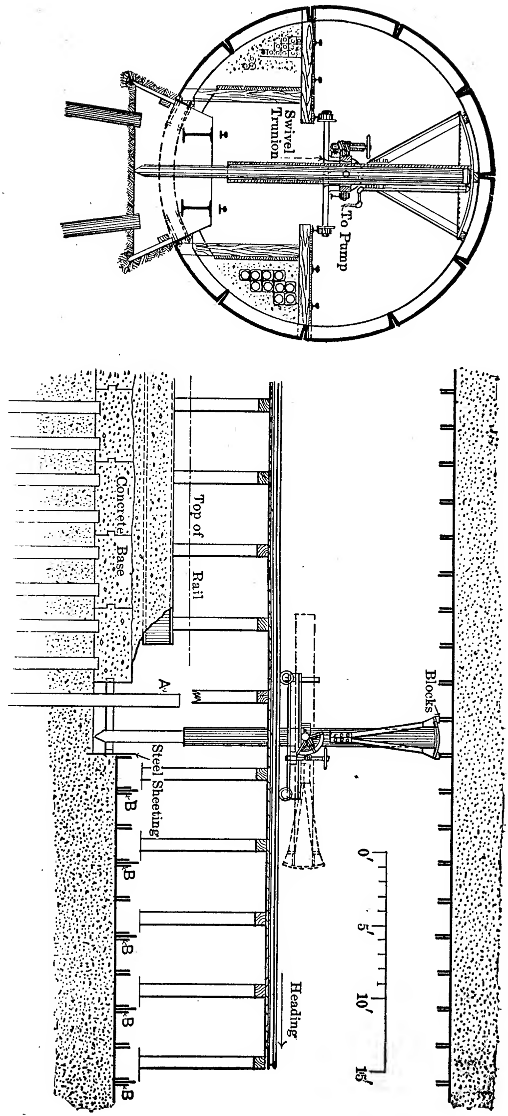


FIG. 124.—PROPOSED SYSTEM FOR DRIVING WOODEN PILES BY HYDRAULIC PRESSURE TO SUPPORT THE NORTH RIVER TUNNEL OF THE PENNSYLVANIA R. R.
(Designed by J. W. Reno, M. Am. Inst. M.E.)

to an average depth of 60 ft. Experiments show that long cast-iron piles loaded with 100 tons have a compression in the pile itself of $\frac{1}{4}$-in., but the pile recovers after removing the load. A full description of the proposed method is found in the *Eng. News*. It will be sufficient in this volume to show the partially completed work as suggested by Mr. Reno (see Fig. 124).

STEEL SHEET-PILING.

IN Fig. 125 is shown a form of steel sheet-piling made up of structural shapes. The alternate piles are of different sec-

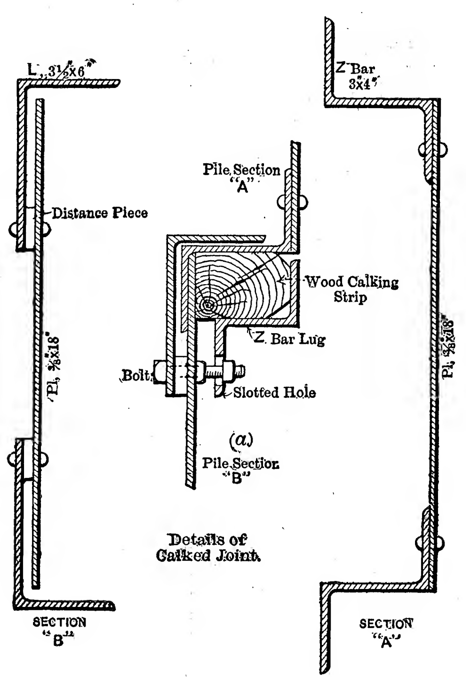


FIG. 125.—STEEL SHEET-PILING. (Henry Wittekind, Inventor.)

tion. Those of section "A" are composed of a $\frac{3}{8}$-in. plate 18 ins. in width, with a Z bar, 3×4 ins., riveted along each edge. Those of section "B" are composed of a similar plate with an angle-iron, 6×3$\frac{1}{2}$ ins., riveted along each edge, the angles being separated from the plate by distance-pieces so as to leave grooves for the outer flanges of the Z bars of the adjacent piles. Two piles of section "B" put together will form a corner. The plate in section "A" may be curved. The piles are driven by an ordinary pile-driver. In order to form a water-tight joint, a calking-strip of soft wood may be placed as shown in Fig. 215 (*a*), the strip being held by Z-bar lugs 4 ins. long, 5 ft. apart. These lugs are secured by bolts through slotted holes.

There are several other types of interlocking shapes, channels, angle-irons, and plate. For ordinary use the wooden sheet-piling is more economical and more easily handled.

A TILTING PILE-DRIVER.

THE importance of driving the two outside piles of a three- or four-pile bent in trestles on a batter seems to be ignored or neglected. This matter is fully discussed on pages 226 to 235 of this volume. The details of construction of an ordinary pile-driver are shown on page 223. The construction of a tilting pile-driver is very simple. The leads may be pivoted to a strong wooden or iron "A" frame. Another type is shown in Figs. 126 and 127.

It is intended to drive batter-piles at 2 ins. batter per foot. The tilting is effected by means of a bolster on which the heads are placed, having a screw at each end with a ratchet-nut, *N* (see Fig. 126). Men work these screws, and can throw the leads to the desired batter. When the pile is driven below the leads the machine is racked sideways by means of a ratchet-wheel, *W*, connected to a shaft running the entire length of the machine, 48 ft. The shaft is connected to pinions, *P*, which gear into rack-rails. The engine as well as the leads is mounted over the shaft and pinions, the pinions being shouldered with rollers, *R*, which bear on the track. The whole machine is carried forward on dollies, *D*. The driver is equipped with a 2600-lb. hammer operated by a Mundy double-cylinder, double-drum friction-clutch, hoisting-engine with a 7×12-in. cylinder and

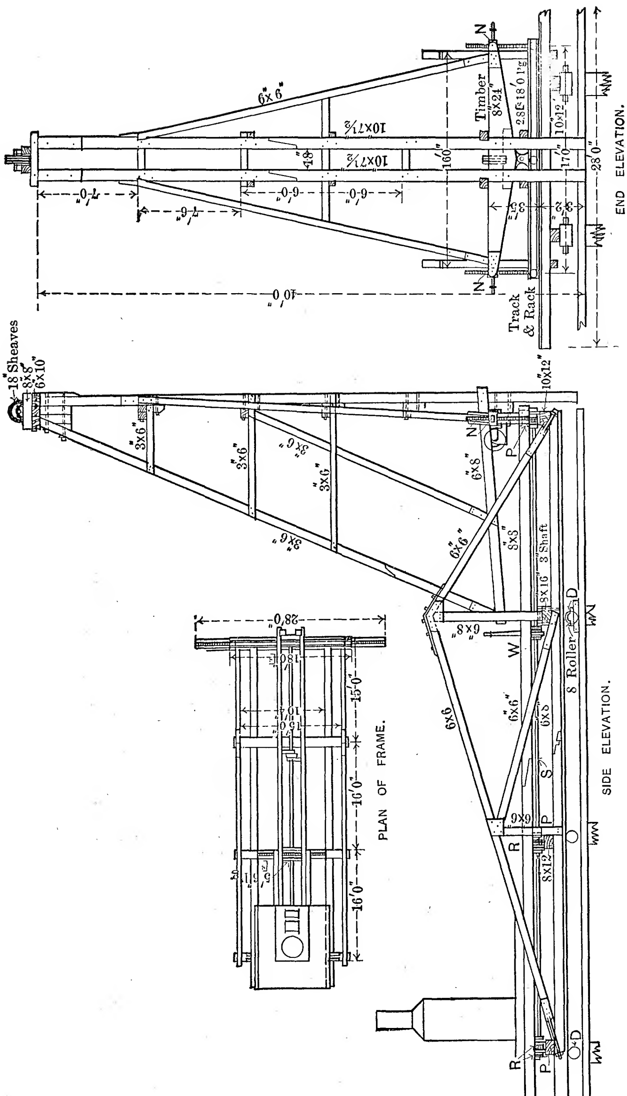


FIG. 12.6—A NOVEL TILTING PILE-DRIVER.

large boiler. The hammer falls on a cast-iron cap weighing 400 lbs. The table on page 504 shows the number of blows of the hammer to drive each pile of four four-pile bents. The piles *A* and *D* were driven on a batter of 2 ins. per foot. Piles

FIG. 127.—DRIVING A BATTER-PILE FOR A TRESTLE-BENT.

B and *C* are plumb-piles. The table shows that batter-piles are driven as easily as plumb-piles. (See articles in *Eng. News* by J. H. Baer.)

TABLE SHOWING NUMBER OF BLOWS TO DRIVE PILES.

Pile-bents.	Length Billed.	Cut Off.		In Structure.		Penetration.	Number of Blows.	Pen. under Last 5 Blows.	Average drop of Hammer.
	Ft.	Ft.	Ins.	Ft.	Ins.	Ft.		Ins.	Ft.
24A	26	2	0	14	0	10	31	15	25
24B	26	3	0	12	0	11	35	15	30
24C	26	1	8	12	4	12	40	10	30
24D	26	1	8	11	4	13	50	10	30
25A	28	2	0	8	0	18	38	10	30
25B	28	1	9	9	3	17	36	10	30
25C	28	2	0	9	0	17	40	8	30
25D	36	5	0	9	0	22	59	8	30
26A	30	1	8	14	4	14	38	8	30
26B	30	2	0	13	0	15	57	5	30
26C	30	2	0	13	0	15	97	4	30
26D	30	1	8	14	4	14	56	6	30
27A	30	2	0	13	0	15	67	5	30
27B	30	2	0	13	0	15	50	8	30
27C	30	1	10	13	2	15	34	10	30
27D	30	2	6	12	6	15	47	6	30

TESTS FOR SOUNDNESS OF PORTLAND CEMENT.

FOR all purposes of construction the essential qualities of cement are strength and soundness. Any standard brand of cement may be relied upon to have sufficient strength for all ordinary purposes; though, as a protection against an indiscriminate use of the many brands of cement on the market, engineers properly require cements to indicate a certain degree of tensile strength when tested in a machine. Some of these requirements are perhaps higher than necessary. But if engineers are willing to pay the price, they can generally get what they demand. Ordinarily very little stress is laid on the question of soundness or durability. An excess of free lime or a lime not sufficiently hydrated produces unsoundness. This excess of lime may be due to underburning, to lack of seasoning, or to coarseness of grinding.

Some of the tests for soundness are: (1) the boiling test; (2) the steam test; (3) the hot-water test; (4) the kiln test. Of these the more commonly employed are the first three.

In the regular boiling test cakes of cement in the form of a small egg, 1 in. in diameter, are kept in moist air for 24 hours, then placed in cold water which is gradually raised to the boiling-point and maintained at that temperature for three hours. Tests in steam, in hot water, and in combinations of the two are made for purposes of comparison. A cement at first failing in these tests may be found to show very different results if allowed to season for a few weeks, the unsoundness developed in the first test not appearing in a later one. This is indicated in the following table:

EFFECT OF AGE OF CEMENT ON RESULTS OF BOILING TESTS.

Age of Cement when Tested.	Tensile Strength, Neat.			1:3 Sand.		Normal Pat Tests.		Boiling Test.
	1 Day.	7 Days.	28 Days.	7 Days.	28 Days.	In Air 28 Days.	In Water 28 Days.	
1 week	550	765	762	171	225	Curled and soft	Slightly checked	Partly disintegrated
2 weeks	548	767	771	170	246	Slightly curled	Slightly curled	Checked and cracked
3 "	492	718	763	182	244	O. K.	O. K.	Slightly checked
5 "	427	692	747	183	249	O. K.	O. K.	Sound

A long and interesting article by W. P. Taylor in *Eng. News* July 23, 1903, gives a full discussion of this subject with numerous tables and illustrations. The general conclusions drawn are that, although in a great majority of cases the results of the boiling test can be considered as being indicative of the future behavior of the cement from a laboratory standpoint, one cannot be too careful in condemning cements on this test alone, unless his experience has covered a large number of tests and many varieties of material, and even then it is questionable if the rejection of cements on the results of this test alone is justifiable. It is often policy to hold a cement for a week or two, so that it may have an additional amount of seasoning. The laboratory tests are usually made on new cement, while that used in construction is several weeks old. The indications of unsoundness with neat cement are always greater than when mixed with sand. This is observed in the laboratory tests as

TABLE SHOWING TENSILE STRENGTH OF CEMENTS, MIXED NEAT WITH DIFFERENT PROPORTIONS OF WATER.

Cement.	Water Per Cent.	Sieve Test Residue on			Wire minutes.		Tensile Strength. Lbs.					
		No. 50.	No. 100.	No. 180.	Light.	Heavy.	24 Hrs.	7 Days.	28 Days.	3 Mos.	6 Mos.	12 Mos.
Giant Portland	15	0.15	5.4	21.2	12	207	371	655	875	941	720	787
	16	....	....	....	29	297	303	750	973	1008	735	816
	18	....	....	....	80	355	260	649	773	831	645	748
	20	.....	.....	.....	142	402	233	500	693	716	621	676
	22	.....	.....	.....	268	473	184	546	635	658	601	589
	24	.....	.....	.....	327	912	167	539	649	644	629	755
Union Natural	23	0.1	4.6	10.2	13	32	212	251	252	311	275	356
	25	.....	.....	.....	18	39	185	218	215	289	300	341
	27	.....	.....	.....	21	42	150	188	220	257	272	314
	29	.....	.....	.....	20	52	128	178	202	246	248	256
	31	.....	.....	.....	21	57	112	173	199	224	259	309
	33	.....	.....	.....	27	85	104	172	182	267	246	290
	35	.....	.....	.....	38	137	93	121	178	260	286	319
	37	.....	.....	.....	34	160	85	108	168	262	306	326
	39	.....	.....	.....	67	233	85	119	202	252	371	400
Atlas Portland	13	0.1	7.0	18.0	13	270	366	775	859	1067	892	832
	14	.....	.....	.....	18	303	404	780	891	972	852	781
	16	.....	.....	.....	22	327	363	602	725	844	806	723
	18	.....	.....	.....	15	383	308	570	723	785	728	724
	20	.....	.....	.....	56	703	225	590	718	760	674	636
	22	.....	.....	.....	52	833	166	554	649	731	643	604
	24	.....	.....	.....	188	918	42	510	691	695	632	574
Hoffman Rosendale	23	2.3	12.4	21.9	22	59	138	177	271	332	284	264
	24	.....	.....	.....	...	78	125	141	264	342	309	310
	25	.....	.....	.....	35	120	150	164	216	308	318	321
	27	.....	.....	.....	49	143	117	116	194	305	345	272
	29	.....	.....	.....	76	166	96	105	164	272	320	267
	31	.....	.....	.....	117	212	72	72	159	270	371	225
	33	.....	.....	.....	115	235	62	71	147	277	379	244
	35	.....	.....	.....	127	400	50	64	112	245	318	315
	37	.....	.....	.....	198	828	59	62	96	...	284	351
	39	.....	.....	.....	260	1057	54	56	85	...	355	364

Note.—Results shown are the averages of six briquettes made.

TABLE SHOWING TENSILE STRENGTH OF CEMENT MORTAR WITH SAND GRAINS OF DIFFERENT DIAMETERS.
Results given are the average of six briquettes.

Sand Gage, Per Cent Used.				Natural-cement Mortar 1:1.								Portland Mortar 2:1.			
					Union.				Hoffman.				Giant.		
No. 30.	No. 20.	No. 100.	Fine.	Water, Per Cent	7 Days.	28 Days	6 Mos.	Water, Per Cent	7 Days.	28 Days.	6 Mos.	Water Per Cent	7 Days.	28 Days.	6 Mos.
100	....	....	...	17	156	193	352	15	115	163	314	10½	286	288	412
	100	....	...	17	151	194	349	15	118	146	286	10½	294	331	473
		100	...	17	153	187	340	15	91	110	257	10½	201	226	294
			100	17	100	123	307	15	71	76	186	10½	129	159	223
80	10	10	...	17	154	210	358	15	94	124	301	10½	361	380	486
70	15	12½	2½	17	142	190	332	15	86	107	254	10½	301	303	428
60	20	15	5	17	143	192	342	15	83	107	285	10½	307	311	419
50	25	17½	7½	17	140	208	345	15	80	89	291	10½	391	400	538
40	30	20	10	17	133	197	362	15	90	82	296	10½	350	355	475
30	25	30	15	17	123	191	329	15	78	77	266	10½	362	359	478
20	20	40	20	17	128	199	318	15	66	73	285	10½	317	374	480
10	15	50	25	17	122	201	324	15	68	72	221	10½	291	354	488
50	..	..	50	17	108	193	317	15	62	70	239	10½	247	287	351
50	50	..	..	17	154	222	323	15	82	107	316	10½	440	408	542
50	..	50	..	17	150	210	344	15	78	88	290	10½	309	336	438
25	25	25	25	17	125	183	302	15	74	68	250	10½	279	337	447
	Crushed Quartz.						3 Mos.				3 Mos.				3 Mos.
40	..	60	..	16	179	256	355	14	93	100	142	9½	257	331	351

All proportions and percentages determined by weight. Natural sand used, first passed through No. 8 screen and residue excluded. No. 30 sand passed No. 20 screen and caught on No. 30 screen; No. 20 sand passed No. 8 screen and caught on No. 20 screen, No. 100 sand passed No. 30 screen and caught on No. 100 screen; "Fine" is clean white sand sifted through the No. 100 screen.

well, neat-cement briquettes disintegrating completely, but retaining their strength in the sand tests.

Tables showing tensile strength of cements and of cement mortar are given on pages 505 and 506. Both are taken from *Eng. News*, Aug. 6, 1903.

RELATIVE STRENGTH OF WET AND DRY CONCRETE.

FORTY-FIVE 6-in. cubes were made with three percentages of water and broken in a testing-machine at the age of 7 days, 30 days, and 90 days. For dry concrete 6 p. c., for medium 7.8 p. c., and for wet 9.4 p. c. of water were employed. The concrete was 1 vol. loose cement, Chicago AA Portland, 3 vols. sand. The sand contained a small per cent of gravel, 6 vols. of broken limestone, of size passing a 1-in. mesh. The dry mortar when squeezed would retain its shape, but under tamping in free water would flush to the surface; when the medium was tamped the concrete would quake and water would come to the surface; the wet quaked in the hand and would not admit of much tamping. The conclusions drawn from these experiments are as follows:

(1) Dry concrete should never be used under any circumstances.

(2) Medium concrete may be used where immediate strength is desired.

(3) Wet concrete is stronger than either dry or medium at any age over three months.

(4) Wet concrete is more elastic than dry, as shown by distortion, at failure.

(5) A compact mass is obtained with little tamping when wet, while no amount of tamping will give a compact mass with dry mixtures.

(See article by James W. Sussex, *Eng. News*, July 16, 1903.)

TABLE OF CRUSHING STRENGTH OF 6-IN. CUBES.

Age Days.	Plasticity.	Tamping.	Crushing Strength in Pounds.	Mean.
7	Dry	Light	46,800	
7	"	"	41,700	43,400
7	"	"	41,800	
7	Medium	"	86,000	
7	"	"	80,600	82,400
7	"	"	80,000	
7	Wet	"	38,300	
7	"	"	36,600	37,600
7	"	"	37,900	
7	Dry	Hard	46,300	
7	"	"	46,500	48,300
7	"	"	52,000	
7	Medium	"	46,800	
7	"	"	51,100	47,800
7	"	"	45,500	
30	Dry	Light	55,000	
30	"	"	63,000	63,000
30	"	"	71,000	
30	Medium	"	86,600	
30	"	"	80,000	82,400
30	"	"	80,600	
30	Wet	"	85,200	
30	"	"	79,500	80,300
30	"	"	76,300	
30	Dry	Hard	67,400	
30	"	"	70,800	70,600
30	"	"	73,800	
30	Medium	"	95,200	
30	"	"	91,800	92,300
30	"	"	90,000	
90	Dry	Light	87,100	
90	"	"	98,400	89,800
90	"	"	94,000	
90	Medium	"	76,000	
90	"	"	76,300	77,400
90	"	"	80,900	
90	Wet	"	110,700	
90	"	"	110,400	109,300
90	"	"	106,700	
90	Dry	Hard	90,000	
90	"	"	92,000	93,600
90	"	"	98,400	
90	Medium	"	99,300	
90	"	"	90,500	93,100
90	"	"	89,500	

Taking the 90 days' test and dividing 36 sq. ins., the area of the cubes, we have:

	Dry.	Medium.	Wet.
Strength per square inch........	2578	2150	3040 lbs. per sq in.

RUBBLE CONCRETE.

In this volume and in his Treatise on Civil Engineering the author has repeatedly recommended the use of rubble concrete, whether of large or small stones. He recently built a storage dam, 30 ft. in height, entirely of go-as-you-please rubble, the requirements being simply to place good-sized stones on the faces, that all stones should be well bedded in 1:3 mortar with Portland cement, and that all side joints should be filled with mortar and small stones, but required in all cases that the smal. stones should be driven in mortar, and in no case were the small stones to be placed first and plastered over with mortar, leaving the interstices unfilled. In *Eng. News*, July 16, 1903, is found an editorial upon this subject, from which the following is transcribed. In this description the joints are supposed to be filled with concrete. Rubble concrete is defined as a mass of masonry for walls composed of large rubble stones and the joints filled with wet concrete. For small walls and foundations it is better to ram spalls, or one-man stones, into a layer of wet concrete Concrete possesses three distinct advantages over ordinary stone masonry: (1) it does not require skilled stone-cutters to cut it, (2) nor skilled masons to lay it; (3) it may be made of shaly stone or gravel. The disadvantages are: (1) it requires the installation of a stone-crushing plant; (2) it usually requires frames or moulds; (3) it requires more cement per cu. yd. than any other class of masonry. For small walls the cost of quarrying and cutting dimension stones controls the cost of the work; in large walls and dams, piers and abutments, the cut stone is a small per cent of the total yardage, and the backing or filling largely controls the cost. Second-class retaining-walls on the Erie Canal required 0.6 barrel of Portland cement per cu. yd. of masonry, 1:2 mortar. Whereas a 1:2:5 concrete in place consumed 1.1 barrel of cement per cu. yd. with cement at $1.60 per barrel, there is a saving in favor of rubble of 80 cts. per cu. yd. The forms cost about 50 cts. per cu. yd., and crushing the stone 30 cts. Placing cost of laying rubble at 80 cts., and that of mixing and laying concrete at 40 to 60 cts., if mechanical mixing or hand-mixing is employed, there is an ultimate saving in favor of rubble of $1.20 to $1.40 per cu. yd. It is claimed that at the same cost rubble is stronger than concrete, as a richer mortar may be used in rubble. The comparison is not intended to cover the light reinforced walls.

FOUNDATIONS FOR THE BUILDING OF THE NEW YORK STOCK EXCHANGE.

(See *Eng. News*, Sept. 26, 1901.)

The foundations for this building were carried to rock at a depth of 60 ft. below curb-line. The material penetrated is a fine micaceous quicksand, water-bearing, with some clay strata. As it was intended to utilize the entire underground space, the contract required continuous side walls impervious to water and forming a water-tight joint with bed-rock, and a concrete floor covering the rock. Similar structures have been built by sinking pneumatic caissons with about 1 ft. clearance between the several caissons, and filling the space between the two with packed clay in order to secure a water-tight joint. Under the Stock Exchange Building it was determined that a continuous concrete foundation should be built under the outside walls. The method adopted is as described below.

Narrow pneumatic caissons rectangular in shape, 30 ft. in length and 8 ft. wide, outside measurement, were built of wood, and sunk separately end to end and as close together as practicable. The walls of the caissons are built of vertical timbers 4 ins. in thickness and thoroughly calked. They were strongly braced together by angle-irons. The several sections of caisson were connected by male and female angle-irons. The angle of the upper section slips down below the top of the lower section, and the two angles are bolted together. A water-tight joint was made with a rope gasket dipped in linseed-oil. The sections of caisson were each built complete for each story, and handled by means of powerful boom derricks. Each caisson weighed about 15 tons. Special iron frames were connected with the caissons to which the hoisting-ropes were attached. The working chamber was formed by iron roofs placed in the lower sections. The usual and necessary working-shafts and air-shafts were provided. The sinking was done in the usual manner, and weight provided by filling above the roof with concrete. Iron forms of semi-elliptical section, 4×5 ft. in dimensions, are set in each end of each section in order to leave a well between the

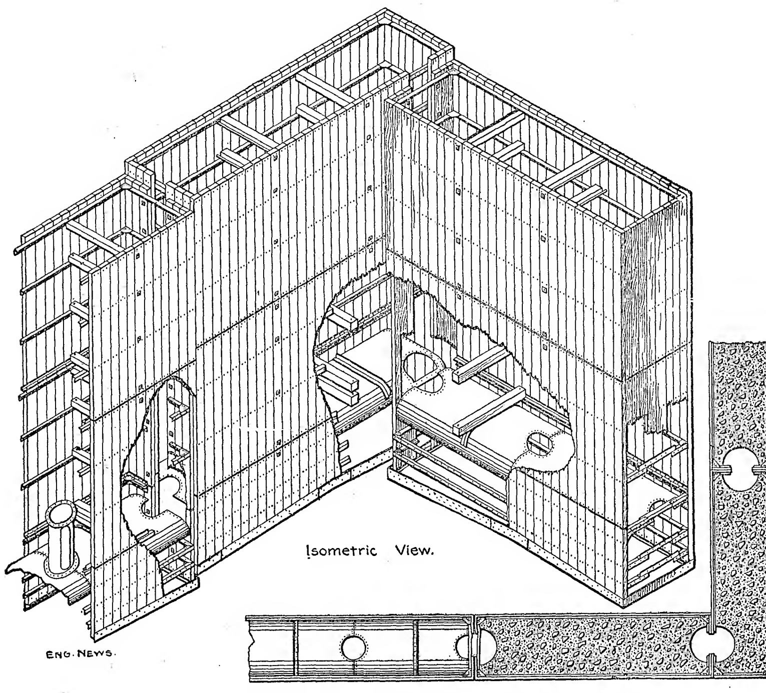


FIG. 128.—ISOMETRICAL VIEW ILLUSTRATING THE PROCESS OF SINKING CONTINUOUS CAISSON FOUNDATIONS.

adjacent ends of section, and arrangements were provided for ultimately removing the ends of the caissons and filling the elliptical well, one half of which was in each end of adjacent caissons, with concrete, thus completing the continuity of the concrete wall. In this manner each caisson of 30 ft. length was sunk to rock, a tight joint was made between the bed-rock, and the filling of the working chamber proceeded. As the lower or bottom section of each caisson was sunk another section was placed on top and filled with concrete, the same arrangements and provisions for forming a continuous wall being made. When the caissons reached bed-rock preparations were made for connecting the two adjacent caissons with concrete. The ends of the caissons were provided with doors bearing against bevelled seats in the caisson wall and secured with bolts. When everything was ready the bolts were removed and the door in the end of one caisson removed by pulling it into the air-chamber. Men rapidly filled the open space between the two caissons with prepared clay balls, which were held in place by the air-pressure. The door in the end of the adjacent caisson was removed in a similar manner, and the open elliptical well between caissons filled with concrete. Before filling the well the adjacent walls of two caissons were forced together by means of strong bolts, and water-tight joints were formed by wedging, if necessary. An isometrical view of some of the separate caissons placed together near one corner is shown in Fig. 128. This shows the construction of the caissons and their relative positions after having been sunk to bed-rock.

While caissons for the outside walls were being sunk, circular caissons, also built of wood, were sunk to rock at intervals in the enclosed areas. These cylinders are $6\frac{2}{3}$ ft. in diameter. Having reached rock, the bottom is thoroughly sealed with concrete. The footings for the columns were placed in the bottom of the cylinders (see Fig. 129). Finally the entire mass of earth enclosed was excavated and a flooring of concrete placed over the rock. The enclosing walls, having for all time to support the earth-pressure, are strongly braced with iron beams. Horizontal bearing-beams were set against the walls of the caissons, and by means of hydraulic jacks the struts abutting against these beams were put under an initial compression greater than that exerted upon the walls from the outside, considering them as retaining-walls and as sustaining hydrostatic pressure.

As the walls had to be impervious to water, and to provide against any possibility of there being unfilled spaces or cracks in the concrete formed during the process of settling, it was arranged to force grout through the entire mass. The roof of the air-chamber was removed and placed on top of the shafts; a liquid grout was introduced and compressed air turned into the shaft, thereby forcing the grout into and through any cracks or openings in the concrete.

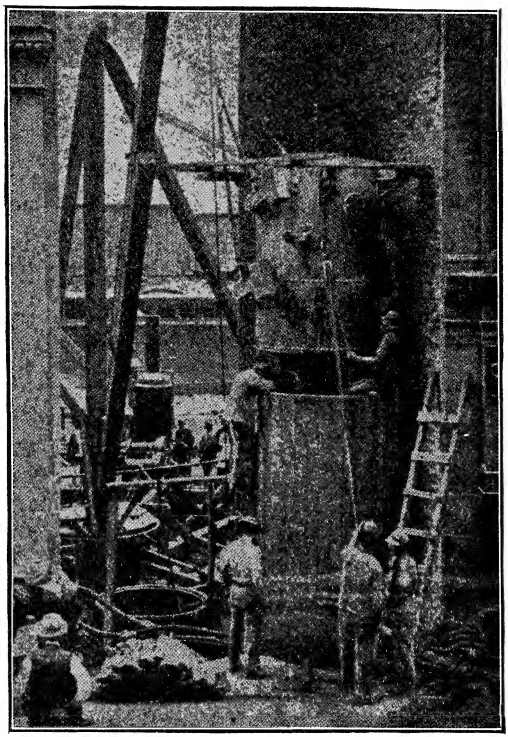


FIG. 129.—CYLINDERS BEING SUNK FOR FOUNDATIONS IN THE NARROW WALL STREET EXTENSION OF THE NEW STOCK EXCHANGE BUILDING.

The air-chamber gang consisted of eight men and one foreman. The material was shovelled into bottom-dump buckets, 30 ins. in diameter and 36 ins. high, and was hoisted through the air-locks. In removing the quicksand the men averaged about 10 cu. yds. per hour. The maximum rate of sinking a caisson was 4½ ft. in four hours. Mr. John F. O'Rourke was the consulting and contracting engineer.

REPAIRING THE FOUNDATIONS OF PIERS BUILT IN ARKANSAS RIVER, AT LITTLE ROCK, ON THE LINE OF THE MISSOURI PACIFIC R. R.

ABOUT the year 1898 the author of this volume was called to Little Rock to make necessary examinations and to furnish plans for remedying a serious condition of settlement and careening of several piers of the bridge over the Arkansas River. The piers had been built a number of years prior to the above date, but the careening had been gradual and continuous and had become so serious that continued use of the structure seemed impracticable. All of the piers rested upon pneumatic caissons made of wood and filled with concrete or broken stone. It was impossible to obtain any reliable, accurate, or definite information in regard to the material on which the caissons rested, or as to the care in and character of the construction. General designs of the caisson were found in the engineer's office. The caisson for the pivot-pier is square in horizontal section, and was found to be set diagonally relatively to the axis of the river; the other caissons are all rectangular in section. The design was the one commonly used at that time, resembling the designs used exclusively by Mr. George S. Morrison (see page 296 of this volume).

Attempts had been made to check the settling and careening of the caissons by driving a large number of piles around the piers and depositing large quantities of riprap. This had accomplished little or no good, but increased greatly the difficulties of making examinations as well as performing the subsequent work of repairs.

The author sunk a number of 3-inch pipes to bed-rock. This work was rendered very difficult by encountering large riprap stone. He, however, succeeded in getting a sufficient number of pipes down to bed-rock, which was found on quite a slope under the caisson. This led to the inevitable conclusion that the caisson only rested on rock at the down-stream angle, and rested on sand under a large portion of the periphery of the caisson. The pipes were sunk with the water-jet, aided by a heavy wooden hammer. The idea had been advanced that the settling was caused by the timbers having rotted. To disprove this, pipes were let down at a number of places on to projecting ends of timbers, and by means of augers welded to the pipes bore-holes were forced into or through the timbers. In every case the pieces of wood brought up were found to be sound, fresh, and clean in appearance.

There seemed but one thing to do, namely, to sink a narrow pneumatic caisson around the piers, leaving between the outside of the inner wall of the new annular and the old caisson a space of about 4 to 5 ft. Weight for sinking was a clay puddle placed in the crib above the pneumatic caisson proper, and if this should not give sufficient weight, it was intended to supply the deficiency with old rails. After reaching bed-rock the working chamber was to be filled with clay puddle. The purpose was to convert the pneumatic caisson and crib above into a coffer-dam, pump out the water between the new and the old caisson, and remove the sand and riprap to bed-rock. In this space a solid wall of concrete, around the old pier or around a good portion of it, was built. Upon this base thus formed a cylinder of iron was placed enclosing the masonry of the round pier, but itself placed in a vertical position and centred on the line. The irregular space between the iron cylinders and the masonry of the pier was filled with concrete. On this substructure the circular rack and pivot of the swing-bridge was to be placed in its proper position. The caisson for the pivot-pier was constructed of wood mainly, square in plan, four sections, about 40 ft. in length and 8 ft. in width between the two walls, forming the sides. At the corners a connection was made between adjacent faces by means of a deep channel section fastened to one side, and a square timber fastened to the other side and sliding in the channel. This annular caisson was to be sunk as a whole, or each face separately, as should be found most convenient. The crib above the caisson was held down by hook rods. After completion of the work and loosening the hook rods and a few bolts at the corners, the cribs could be readily removed, which was necessary in order not to narrow the navigation channels on either side of the pivot pier. The construction of the sections of the caisson was in most respects so much like that of the sections of caissons for the New York Stock Exchange Building previously described that no drawings are necessary (see Fig. 128), although it is true that they differed in many minor details.

COST OF CONCRETE TUNNEL-LINING AND OF TUNNEL EXCAVATION.

THE foundation trenches were excavated from 1 to 3 ft. deed and levelled up with concrete composed of 1:2:4 mixture, the stone being the run of the crusher and of the size to pass 1-in. ring. The foundation contained 200 cu. yds.

The side- or bench-wall forms were constructed resting on the concrete foundation. The forms were built in 12-ft. sections, with plates and sills 4×6 ins., and studs 4×4 ins., of hemlock, spaced 3 ft. centres. The sheathing was 2-in. dressed and matched spruce. The forms were well braced in front; the outer or back face was supported by means of spalls filling a space between the form and a 1-in. sheeting resting against the sides of the tunnel. The space filled with spalls was intended to collect the seepage-water and carry it to weep-holes placed at intervals through the concrete side walls. The forms were filled with concrete to the springing-lines of the arch. Those forms for side wall were removed the day after placing the concrete, and the concrete face was rubbed smooth with wooden floats. At the centre of the tunnel arch forms were erected over a distance of 99 ft., the lagging laid in 12-ft. lengths. As the concrete arch was completed, the 12-ft. forms were moved and carried to the ends, using the same forms over a second time. The concrete was dumped from barrows or shovelled into the forms as was found convenient or necessary. The arch forms were removed in a short time after placing the concrete. In one case a form was lowered in 90 hours after the section had been keyed; there was no evidence of settlement at the crown. The arch ring was waterproofed by means of six layers of tar paper laid in hot tar over the extrados of the arch. The space above the arch ring was filled with spalls packed.

The bench wall contained 692 cu. yds. and the arch ring 932 cu. yds. of concrete, all of 1:2:4 mixture. The portal spandrel walls

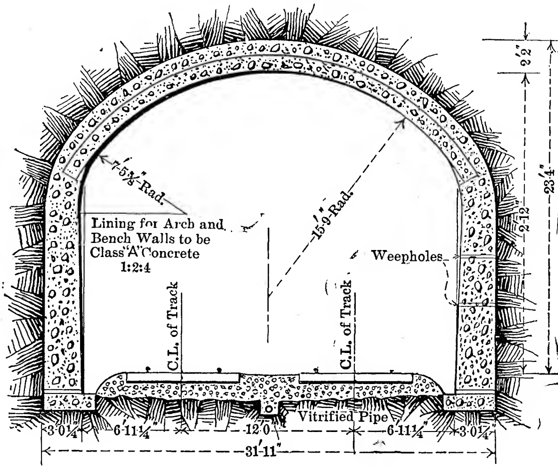


FIG. 130.—CROSS-SECTION OF PEEKSKILL TUNNEL, SHOWING LINING.

were of 1:3:6 concrete. The tunnel was 275 ft. from portal to portal. The following table gives the itemized and total cost:

Cement, at $1.63		$5,755.50	
Sand, at 75 cts.		662.94	
Crushed stone, at 80 cts.		1,303.20	
Lumber:			
Mixing-platforms and runways	$336.89		
Ribs, including band-sawing	234.10		
Backing-boards	134.44		
Lagging	341.04		
Sheathing	268.49		
Plates, sills, studs, and braces	182.75		
		1,497.71	
Coal		118.73	
Oil		16.12	
Hardware, nails, spikes, etc.		224.39	
Tools		181.10	
Freight on stone, cement, etc.		3,089.86	
Labor, including superintendent, foreman, timekeepers, etc.		8,036.31	
			$20,885.76

Average cost per cu. yd., $10.72.

The following table gives the cost of rock excavation in tunnel and open-cut excavation. The rock lay in strata inclined at an angle of 45°. As a result 7791 cu. yds. were removed, while only 7028 cu. yds. were paid for. (See article by G. W. Lee, *Eng. News*, Dec. 17, 1903.) In Fig. 130 is shown a section of arch, in Fig. 131 the end or portal entrance, and in Fig. 132 a part longitudinal section, showing connection between concrete arch and portal masonry.

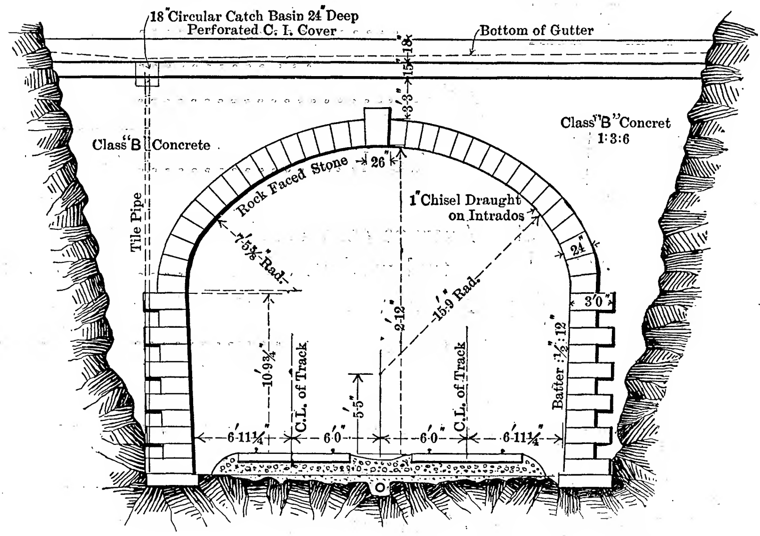


FIG. 131.—ELEVATION AND SECTION OF PEEKSKILL TUNNEL PORTAL.

TUNNEL.

Equipment (less present value) supplies and repairs...	$2,893.52
Dynamite and exploders	1,604.58
Coal	570.80
Oil, waste, etc	92.80
Lumber for office, powder-houses, blacksmith-shops, etc.	129.88
Miscellaneous	92.10
Labor	22,212.86
Total	$27,596.54
Average cost, considering actual yardage taken out	$3.54
Average cost, considering yardage paid for	3.93

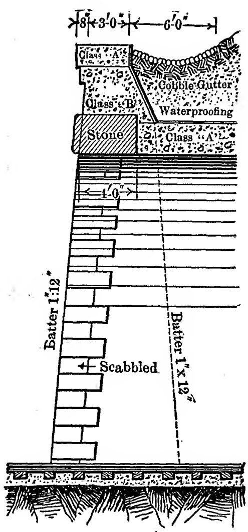


FIG. 132.—LONGITUDINAL SECTION OF PEEKSKILL TUNNEL.

TUNNEL APPROACHES AND ROCK CUTS.

Equipment (less present value) supplies and repairs	$11,673.60
Dynamite and exploders	6,588.82
Coal	2,490.13
Oil, waste, etc.	370.59
Lumber for office, powder-houses, blacksmith-shops, etc.	634.22
Miscellaneous	373.19
Labor	69,550.66
Total	$91,681.21
Average cost, considering actual yardage taken out	$2.01
Average cost, considering yardage paid for	2.24

COFFER-DAM FOR PUMPING STATION AT CHICAGO.

THE coffer-dam constructed at Chicago in 1893 was perhaps the largest structure of the kind ever built; it extended about 500 ft. into the lake and was 190 ft. in width. The average depth of excavation was 24 ft., and the footings were excavated to a depth greater by 7 ft., reaching hard-pan. There was about 25,000 cu. yds. of concrete to be placed. For the outside walls there were driven two rows of guide-piles and a sheeting of the Wakefield sheet-filing. The filling was of clay. Over the enclosed area piles were driven, at 16-ft. intervals, in rows, and capped. This constituted the top bracing. Water was pumped out until the surface was lowered 9 ft. and another set of timbers placed. By this means braces were extended over the great distance between the outer walls; otherwise there was nothing different from the common practice in constructing coffer-dams

NEW YORK CITY REGULATIONS FOR STEEL-CONCRETE CONSTRUCTION.

1. The term "concrete-steel" in these Regulations shall be understood to mean an approved concrete mixture reinforced by steel of any shape, so combined that the steel will take up the tensional stresses and assist in the resistance to shear.

2. Concrete-steel construction will be approved only for buildings which are not required to be fireproof by the Building Code, unless satisfactory fire and water tests shall have been made under the supervision of this Bureau. Such tests shall be made in accordance with the Regulations fixed by this Bureau and conducted as nearly as practicable in the same manner as prescribed for fireproof floor fillings in Section 106 of the Building Code. Each company offering a system of concrete-steel construction for fireproof building must submit such construction to a fire and water test.

3. Before permission to erect any concrete-steel structure is issued, complete drawings and specifications must be filed with the Superintendent of Buildings, showing all details of the construction, the size and position of all reinforcing-rods, stirrups, etc., and giving the composition of the concrete.

4. The execution of work shall be confided to workmen who shall be under the control of a competent foreman or superintendent.

5. The concrete must be mixed in the proportion of one of cement, two of sand, and four of stone or gravel; or the proportions may be such that the resistance of the concrete to crushing shall not be less than 2000 lbs. per sq. in. after hardening for 28 days. The test to determine this value must be made under the direction of the Superintendent of Buildings. The concrete used in concrete-steel construction must be what is usually known as a "wet" mixture.

6. Only high-grade Portland cements shall be permitted in concrete-steel construction. Such cements, when tested neat, shall, after one day in air, develop a tensile strength of at least 300 lbs. per sq. in.; and after one day in air and six

days in water shall develop a tensile strength of at least 500 lbs. per sq. in.; and after one day in air and 27 days in water shall develop a tensile strength of at least 600 lbs. per sq. in. Other tests, as to fineness, constancy of volume, etc., made in accordance with the standard method prescribed by the American Society of Civil Engineers' Committee may, from time to time, be prescribed by the Superintendent of Buildings.

7. The sand to be used must be clean, sharp grit sand, free from loam or dirt, and shall not be finer than the standard sample of the Bureau of Buildings.

8. The stone used in the concrete shall be a clean, broken trap rock, or gravel, of a size that will pass through a $\frac{3}{4}$-in. ring. In case it is desired to use any other material or other kind of stone than that specified, samples of same must first be submitted to and approved by the Superintendent of Buildings.

9. The steel shall meet the requirements of Section 21 of the Building Code.

10. Concrete-steel shall be so designed that the stresses in the concrete and the steel not shall exceed the following limits:

	Lbs. per Square Inch.
Extreme fibre stress on concrete in compression.......	500
Shearing stress in concrete.........................	500
Concrete in direct compression......................	350
Tensile stress in steel.............................	16 c00
Shearing stress in steel............................	10,000

11. The adhesion of concrete to steel shall be assumed to be not greater than the shearing strength of the concrete.

12. The ratio of the moduli of elasticity of concrete and steel shall be taken as 1 to 12.

13. The following assumption shall guide in the determination of the bending moments due to the external forces: Beams and girders shall be considered as simply supported at the ends, no allowance being made for continuous construction over supports. Floor-plates, when constructed continuous and when provided with reinforcement at top of plate over the supports,

may be treated as continuous beams, the bending moment for uniformly distributed loads being taken at not less than $\frac{WL}{10}$; the bending moment may be taken at $\frac{WL}{20}$ in the case of square floor-plates which are reinforced in both directions and supported on all sides. The floor-plate to the extent of not more than ten times the width of any beam or girder may be taken as part of that beam or girder in computing its moment of resistance.

14. The moment of resistance of any concrete-steel construction under transverse loads shall be determined by formulæ based on the following assumptions:

(*a*) The bond between the concrete and steel is sufficient to make the two materials act together as a homogeneous solid.

(*b*) The strain in any fibre is directly proportionate to the distance of that fibre from the neutral axis.

(*c*) The modulus of elasticity of the concrete remains constant within the limits of the working stresses fixed in these Regulations.

From these assumptions it follows that the stress in any fibre is directly proportionate to the distance of that fibre from the neutral axis.

The tensile strength of the concrete shall not be considered.

15. When the shearing stresses developed in any part of a concrete-steel construction exceed the safe working strength of concrete, as fixed in these Regulations, a sufficient amount of steel shall be introduced in such a position that the deficiency in the resistance to shear is overcome.

16. When the safe limit of adhesion between the concrete and steel is exceeded, some provision must be made for transmitting the strength of the steel to the concrete.

17. Concrete-steel may be used for columns in which the ratio of length to least side or diameter does not exceed twelve. The reinforcing-rods must be tied together at intervals of not more than the least side or diameter of the column.

18. The contractor must be prepared to make load tests

on any portion of a concrete-steel construction, within a reasonable time after erection, as often as may be required by the Superintendent of Buildings. The tests must show that the construction will sustain a load of three times that for which it is designed without any sign of failure.

FOUNDATION FOR A LARGE GAS-PLANT.

In the spring of the year 1904 the author was requested to visit Dubuque, Iowa, for the purpose of making the necessary examinations and of recommending a proper and suitable foundation for a gas-plant. In that section of the country gas for lighting is used very extensively; large plants are found in nearly all the large cities and in many small ones. As the plants have to be established on low-lying territory, where the soil is more or less soft and silty, the difficulties in the way of securing unyielding foundation-beds are great. Such foundation-beds are, however, necessary, as many of the large portions of the structure must be maintained in a vertical position. Even a slight careening of the holder results in serious inconvenience.

The site of the particular structure to be described is on the bottom bordering the Mississippi River, originally a silty formation overlying a bed of coarse sand, gradually changing to a layer of small gravel overlying a coarse gravel and sand. Over the silt is a layer of city cleanings, refuse matter of all kinds, such as are usually hauled out of a city and deposited where-ever allowed. This made earth or filled-in material is about 10 ft. in depth over the entire low land. In floods the river sometimes rises above the level of the present surface, and low water is some 15 ft., or more, below this surface. Some portions of the bottom were filled with alluvial soil brought from near-by islands. This was a better material and much more uniform and has a more retentive character than the material hauled from the city. The only structure to be described was to be located where the filling had been made of the material taken from the islands. A large number of pits, holes of square section, from 2 to 3 ft. on the sides, were excavated over the surface of the site. The author was satisfied that the permanent moisture line is found at a depth of from three to four feet below the surface. The surface of the water in the river was at the time, and had been for a long period, at least 10 ft. below the surface of the ground. Having satisfied himself on this point, the next step was to sink a number of 3-in. wrought-iron pipes at close intervals around and at interior points on the sight of the holder. These pipes were sunk by means of the water-jet, the water being obtained from a near-by city main, there being a sufficient pressure for the purpose. At the depth of 20 to 25 ft. below the ground surface a layer of coarse sand was reached, and at a uniform depth of 27 ft. a layer of coarse gravel was encountered. In order to find the bearing resistance of the soil at and below the surface, a platform was placed on top of a stick of timber ($12'' \times 12'' \times 10'$) and loaded with pig iron. At a depth of 3 to 4 ft. below the surface the settlement, after remaining in position from 12 to 48 hours and loaded with 4000 lbs. per sq. ft., did not exceed $\frac{1}{2}$ to 1 inch. These points settled, the author commenced work on plans to be submitted to the company. He submitted three plans with estimated cost of each. The holder is $98\frac{2}{3}$ ft. in diameter, and of proper height to hold water 30 ft. in depth. The holder, frame, and permanent parts of the structure are steel. The holder had a flat steel bottom only $\frac{3}{16}$ in. in thickness. The heaviest load on any portion of the structure would not exceed 3000 lbs. per sq. ft. of foundation-bed. The natural soil indicated a very small settlement under 4000 lbs. placed on top of a 12×12-inch timber. Why, it might be asked, not excavate to a depth of 2 to 3 ft. over the entire area, 7700 sq. ft. nearly, fill the excavation with concrete and set the holder on the concrete? (1) It was considered important that there should be no settling at any portion of the area under the holder. There evidently should be no difficulty in constructing an adequate support under the vertical circumferential wall of the holder. The question was submitted to the designer of the holder, what would be a safe deflection of the thin ($\frac{3}{16}$-in.) bottom steel plate of the holder, made of plates riveted to each other and to the bottom of the steel wall by means of angle-irons? The reply was, the bottom of the holder might deflect 6 to 8 ins. without rupture. (2) This was perhaps only an opinion; there might be a defective plate or a few defective rivets. The author knew of no method of making a computation for determining the strength of a circular plate (riveted) 99 ft. in diameter.

Mr. Rankine gives an approximate formula for a thin square plate supported on all four edges. In a late number of *Eng r-ing News* will be found a long and complicated mathematical investigation of this question. This is too cumbersome, intricate, and unreliable for practical use, had the author known of it at the time. It is true, the author had nothing to do with the design of the holder. But he well knew that should any accident occur the fault would unquestionably have been attributed to the foundation.

Determined to take no chances, the author submitted a design in which it was intended to support the entire structure upon concrete columns resting on piles driven into the underlying gravel 27 ft. below the ground surface, the tops of the piles to be cut off at a point 7 ft. below the surface. There were to be 24 large columns of concrete, 8 ft. in diameter, placed around the circumference, spaced about 128 ft. centres; also 60 small columns, 4 ft. in diameter, placed on radial lines, and one cylinder at the centre (see Fig. 133). The total number of piles was 593. Two lines of curved eye-beams rested on the large cylinders, and upon these were supported the circular walls of the holder. All the framework for carrying the sliding parts was riveted to the steel wall. A line of 6-in. eye-beams connected the tops of the smaller cylinders. Radial 10-in. beams were placed, connecting all cylinders and meeting at the centre and finally a concrete filling was to be placed encasing all beams. Upon this structure the whole apparatus was to be placed. This construction was designed to carry the load and give full support to the steel bottom, securing the structure against any possible scouring out of the material in time of floods, and eliminating entirely the question of the safe bearing power of the silt. This construction was estimated to cost some $12,000 to

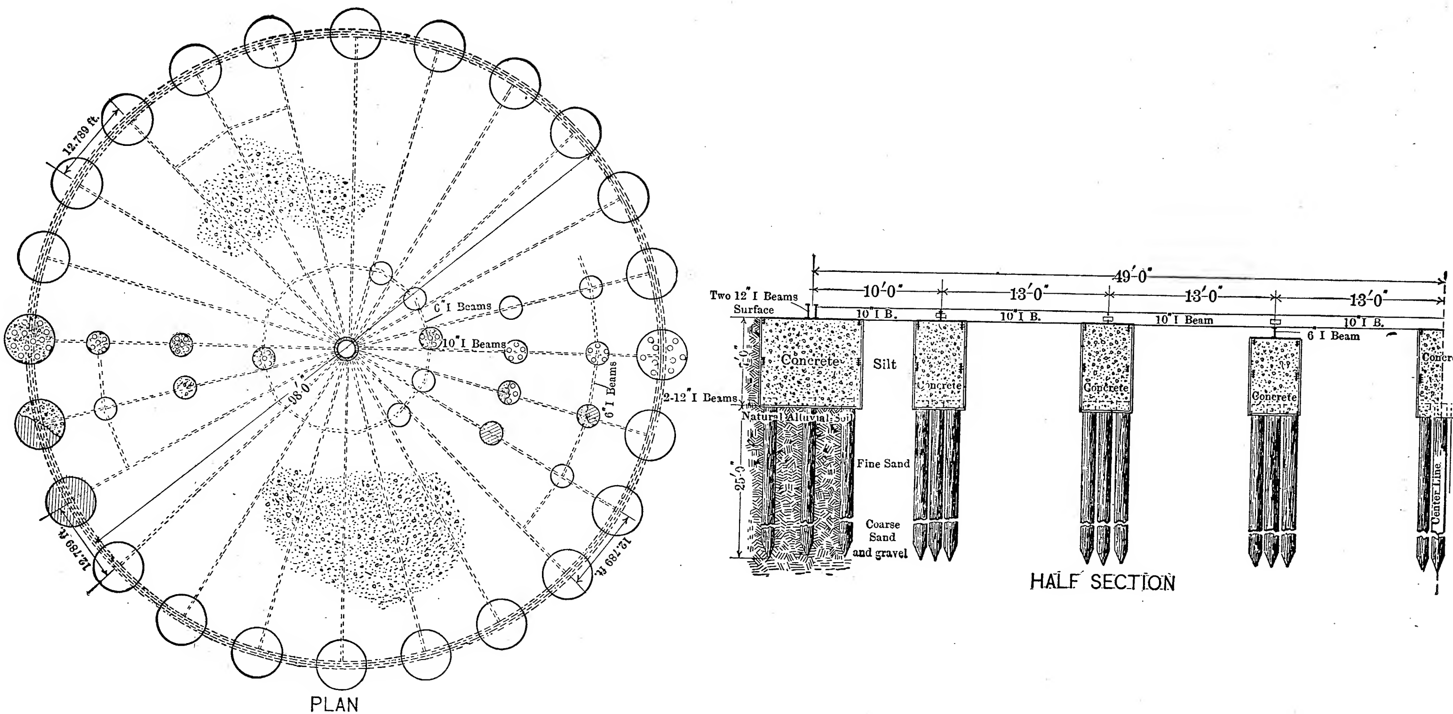


FIG 133.—PLAN FOR HOLDER FOUNDATION FOR KEY CITY GAS CO., DUBUQUE, IOWA.

$15,000. For the support of all other structures, of which there were many, and some of them quite heavy, especially the benches, or structure for the retorts, ordinary wooden piles were recommended, their tops 6 to 7 ft. below the ground surface.

While the author did not think this construction could possibly be protected by patents, there was, in view of the many patents covering almost every kind of concrete-steel construction, some apprehension in the minds of a few persons that there might be lawsuits following. This plan was therefore not adopted. The drawing Fig. 133 will fully show the above construction.

As a second or substitute plan the author proposed excavating a trench 7 ft. in depth under the circumference of the holder, driving a double row of piles spaced $2\frac{1}{2}$ ft. centres each way, and upon these building a concrete wall, 8 ft. wide at bottom, finishing 2 ft. wide at an elevation of 2 ft. above the ground surface. On radial lines, at 4 ft. intervals each way, piles 15 ft. in length were to be driven, cutting them off so that their tops would be about 4 ft. below the ground surface. Resting on these was placed a layer of concrete, 6 ft. in thickness, bonding into the circumferential wall (see Fig. 134). This plan would not cost over half that for the first plan. The author considered it fully equal to the duty required, and as good, or almost as good, as the first plan. The only possible objection that could be raised was whether the tops of the short piles would at all times be below the constant-moisture line. In the author's opinion such would be the case. This plan was adopted, with the single exception that three rows of piles, instead of two, were driven around the circumference.

A third plan was submitted based upon the claim of the designers of the holder that the $\frac{3}{16}$-inch steel bottom would deflect safely 6 or 8 inches without rupture. This plan contemplated a circumferential wall of concrete resting upon three rows of piles. The enclosed area was to be excavated to a depth of 2 to 3 ft., and this excavation to be filled with concrete upon which the holder rested. This left the soil to carry its share of the load. The author did not recommend this plan, and perhaps it was not considered by the board of directors. The plan finally adopted is illustrated in Fig. 134.

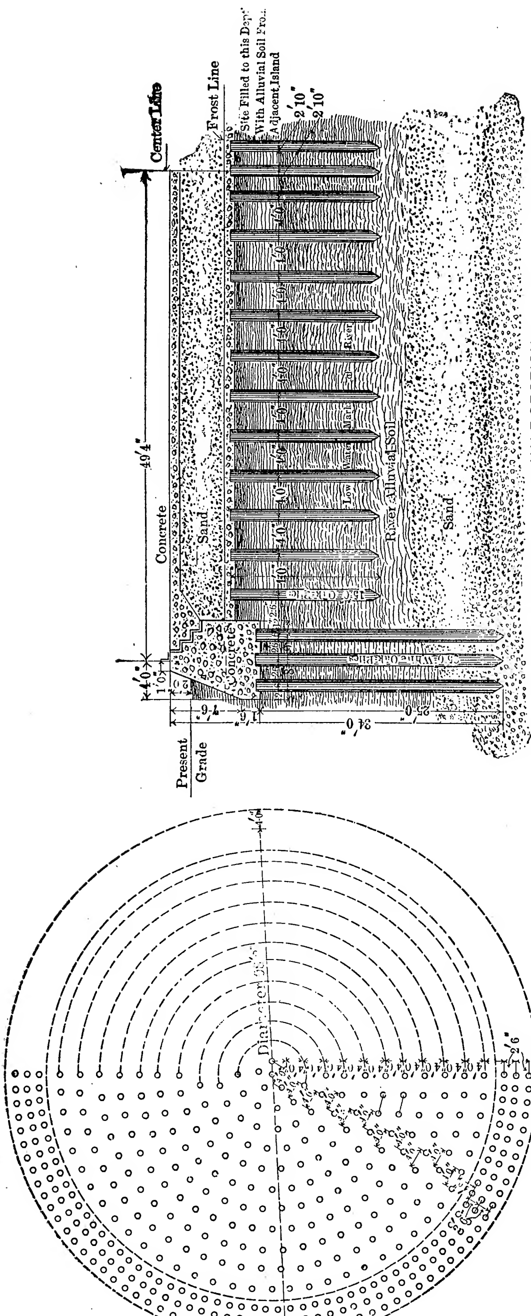


FIG. 134.—PLAN AND SECTION OF FOUNDATION FOR STEEL HOLDER-TANK FOR KEY CITY GAS CO., DUBUQUE, IOWA.

The author's opinion is that all piles are below the permanent-moisture line and therefore will not rot. There is only a remote possibility that the piles, cut off at the depths below the ground surface indicated on the drawing and covered with a thick layer of concrete, will rot even if occasionally the moisture line should fall below their tops when driven in silt or clay. The function of piles is twofold. In the first place they act as a column, to transmit the load to a firm underlying structure, and in the second place they consolidate the soil and thereby increase its bearing power. Admitting that in a long time, and certainly at the end of a very long period, the heads of the piles show signs of deterioration or decay, there will be little or no danger of the structure yielding. The consolidated material will carry the load, there being sufficient beam action in the concrete to carry the load safely.

The author knows of no structure built on wooden piles having failed where the piles are cut off at a depth of from 4 to 10 ft. below the natural surface of the ground, the material being stiff clay or silt, materials very retentive of moisture. The two shore piers of the Cincinnati Southern Ry. bridge at Cincinnati are founded on solid timber platforms resting on piles. The heads of the piles and timber are encased in concrete at an elevation of many feet above low water in the Ohio River. These piers are near to or on the sloping banks of the river. Mr. Bouscaren presented the author drawings of these piers, which were followed in the construction of two shore piers of the Ohio River at Point Pleasant, W. Va. For these piers excavations were made on the sloping banks of the river. The bottom of these excavations was from 15 to 20 ft. above low-water line in the river. Piles were driven in rows $2\frac{1}{2}$ ft. centres each way. They were cut off about 1 ft. above the bottom of the excavation, which was then filled with concrete to the top of the piles. Solid caps 12×12 ins. were placed on the piles, and the intervening spaces filled with concrete. Upon this was laid a solid floor of 12×12-in. timber, on which the masonry rested. Before filling around the masonry with earth, a mass of concrete was built enclosing all exposed timber and extending up the masonry about 2 ft. and packed close to the masonry. This structure was built about the year 1880. The author has never heard of any settling. He does not recommend the timber platform; it is not necessary, adds little to the strength of the foundation, and places a perishable material where, if it should rot, there is little or no possibility of removing the old material and substituting anything better. In another structure, when building a bridge across the Tombigbee River in Alabama, the author left out the timber platform, simply surrounding the heads of the piles with concrete and placing a layer of 2 ft. in thickness of concrete over the piles. The bottom of this excavation was at least 15 ft. above low-water line in the river. These shore piers were built on the sloping banks of the river. The structure was built about the year 1886–87. He has never heard of any settling or other trouble.

While it is unquestionably the wisest and safest plan not to place timber where it may by any possibility be subjected to conditions of alternate wetness and dryness, he believes that wooden piles, whose tops are well below the ground surface and encased in and covered over with concrete, will remain sound for certainly a very long period of time, and the probabilities are that even if a little rot occurs near their tops, no serious harm is likely to be done the structure.

In all of the above cases the author was satisfied that the piles throughout their length were below the moisture-line. In cities where there are numerous lines of underdrains, and leaky sewers, and conduits of many kinds the engineer is not justified in assuming a constant-moisture line above the level of these pipes. Criticisms have very frequently been made of pile foundations in Chicago for the above reason, namely, while the piles may be below the present moisture-line, this line may be lowered at some time in the future. The foregoing remarks apply to silty and stiff clay soils and are not intended to be applied to sandy soils. Thick sand deposits adjacent to streams can have no permanent-moisture line above low-water line in the streams traversing them. In rebuilding an old road in Alabama there were many pile trestles to be renewed. For the most part the bottoms adjacent to the streams were either of a marshy or of a sandy character. Where driven in sand, it was found that the piles, after having been in the ground some twenty years, were more or less rotten to a depth of several feet below the surface. From this fact, together with the fact that long stretches of trestle were built on three piles, and that where four were used, they were all vertical, and it would have been necessary to drive at least two new piles to each bent in order to secure a bearing for the sills of frame trestles, it was determined to drive new piles over the whole length of old road.

For the foundations of the Dubuque gas-plant concrete piles were strongly recommended, and one or more articles appeared in papers condemning the use of wooden piles, on the ground that they would rot, and that they were out of date and no longer employed. The cost of concrete piles is at least five times per linear foot that of wooden piles. Consequently, except under special conditions or where the element of cost is not considered, wooden piles should be employed wherever conditions are favorable.

THE BEARING POWER OF EARTH.

THE usual practice of determining the bearing power of earth by loading a platform supported on a small area, say one or two square feet in area, may and does lead to an entirely misleading and false conclusion. If a 12×12-in. timber having one square foot bearing area is loaded with 4000 lbs. without settling, the conclusion from this that a structure covering 100 sq. ft. will carry safely a load of 400,000 lbs. is not by any means justifiable. In Fig. 135 is shown one square foot of surface loaded with 4000 lbs. Assume the line of shear of the earth to be at an angle of $30°$ with the vertical, then at the depth of 10 ft. below the bottom of the timber the bearing area is $(12.55)^2 = 157.5$ sq. ft., and the original load of 4000 lbs. per square foot is reduced to 25.4 lbs. per square foot. The reduction of the load commences at once: at a depth of 1 ft. the bearing area is 4.62 sq. ft., and the load per square foot at this depth is about 865 lbs. per square foot.

Suppose the bearing area is 100 sq. ft., covering a surface 10×10 ft.; then at a depth of 1 ft. the bearing area is $11.15 \times 11.15 = 124.3$ sq. ft., and assuming a total load of 400,000 lbs., the load per square foot is 3218 lbs., and at a depth of 10 ft. the load per square foot will still be as much as 861 lbs. If then we take a gas-plant covering 7700 sq. ft., about 99 ft. in diameter, the holder containing 14,447,500 lbs., the load per square foot of earth is nearly 2000 lbs. at the surface with the same shear-line of earth; at the depth of 10 ft. below the surface the diameter of the increased

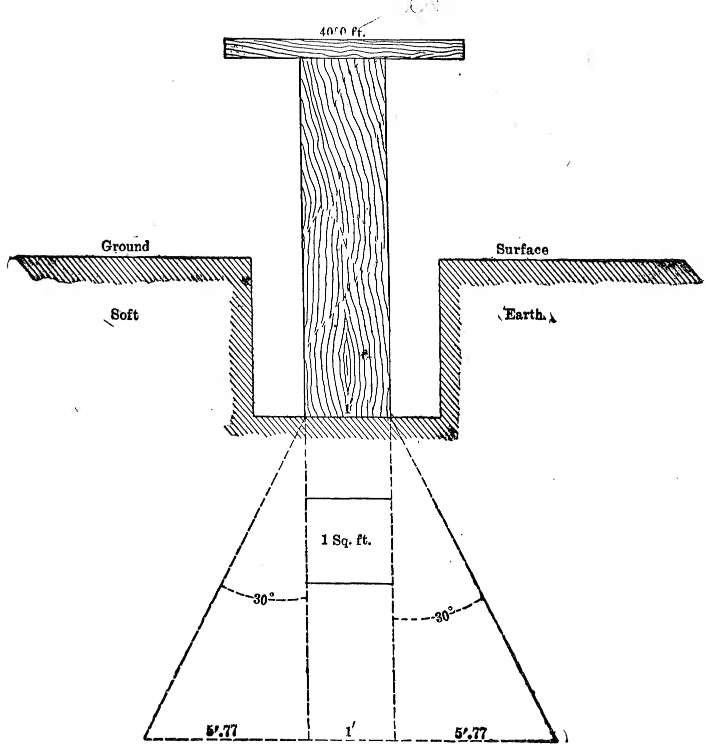


FIG. 135.

bearing area will be 110.5 ft. and bearing area 9600 sq. ft., giving a load of about 1500 lbs. per square foot of area, a small relative reduction of load.

From the above it is seen that the safe load per square foot as determined on a small area of the surface is not a sure indication of the ultimate safe load when distributed over large areas.

INDEX.

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